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Nationwide Urban Runoff Program
Tampa, Florida
Final Summary Report

Tampa Dept. of Public Works, FL

Prepared for

Environmental Protection Agency, Washington, DC

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CITY OF TAMPA

TAMPA, FLORIDA 33602

TAMPA NATIONWIDE URBAN RUNOFF PROGRAM
FINAL SUMMARY REPORT

October, 1983

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October 31, 1983

J-7738

Mr. Ron Giovanelli
City of Tampa
Department of Public Works
City Hall Plaza - 4th Floor
Tampa, Florida 33602

Dear Mr. Giovanelli:

In accordance with our Agreement dated July 6, 1981 and subsequent amendments, we are pleased to submit the Final Summary Report for the Tampa Nationwide Urban Runoff Program Study.

Results and findings of the Tampa Nationwide Urban Runoff Program and Receiving Water Study are presented in five (5) final reports including Appendices and fourteen (14) individual task outputs. Included among the final reports are:

1. Phase I Receiving Water Study,
2. Phase II Final Report,
3. Phase III Stormwater Management Plan,
4. Lower Hillsborough River Report, and
5. Final Summary Report.

This study was conducted by Messers. David R. Bingham and Paul N. Singarella, Ms. Katherine O. Thompson and Ms. Meredith G. Durant of Metcalf & Eddy, under the direction of Dr. Abu M. Z. Alam. Chapters 1, 2, 4 through 7 and part of Chapter 8 of this Summary Report were prepared by Metcalf & Eddy staff. Chapter 3 and part of Chapter 8 were prepared by Priede-Sedgwick staff under Mr. Nilo Priede's supervision.

We thank you for the cooperation and assistance which you have provided to us during the conduct of the work. It has been a pleasure to work for the City on this important project. We look forward to our continued association with the City of Tampa on future assignments.

Very truly yours,

abu m.z. alam

Abu M.Z. Alam
Associate

AMZA:fpc

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We also appreciate the assistance, comments and data provided by personnel at EPA NURP headquarters in Washington, EPA Region IV in Atlanta, the Florida Department of Environmental Protection offices in Tallahassee and Tampa, the USGS office in Tampa, the Tampa NURP Technical Advisory Team, and the Citizens Advisory Committee.

CHAPTER 1

TAMPA NURP STUDY CONCLUSIONS AND RECOMMENDATIONS

The pertinent conclusions and recommendations of the Tampa NURP Study and the Receiving Water Study are presented below:

Phase I Conclusions

1. The lower Hillsborough River was selected as the best receiving water body to study stormwater impacts. Potential water quality problems identified were coliform bacteria, dissolved oxygen and solids levels.
2. Existing data were insufficient to carry out a thorough water quality assessment of the lower Hillsborough River.

Phase II Conclusions

1. The most common storms in Tampa are of short duration and high frequency. These have the potential to wash off large pollutant loadings with a minimum of dilution. Therefore, storms of this type were used for the receiving water quality stormwater impact analysis.
2. Stormwater in Tampa is generally of better quality than that observed in other U.S. cities, with Tampa having below average surface solids buildup.
3. Open bottom inlets are ineffective controls due to excessive maintenance requirements and a lack of storage volume.

4. Drain trenches and ditches are effective at removing hydraulic and pollutant load. Although ditches are less expensive to construct and maintain than drain trenches, their presence is often undesirable.
5. Detention ponds are effective in attenuating hydraulic load and removing pollutant load. Pollutant load removal is highly variable and can range from 0 to 90 percent. When dead storage space is available, better removal of pollutant load occurs. Removal of particulate matter and their associated pollutants is better than removal of soluble pollutants.
6. Flooding will occur in Tampa even during dry years. Flooding primarily occurs during storms with hourly intensities greater than 1 inch/hour.
7. High variability of stormwater quality required bracketing of projected pollutant loads around median values.

Phase III Conclusions

1. Significant flooding is projected to occur during the 5, 25 and 50 year design events. Every feasible flood solution for these design events for each subarea in the lower Hillsborough River drainage basin was characterized.
2. For durations greater than 3 hours, design storms developed using local data from the Tampa International

Airport have larger rainfalls than the Florida Department of Transportation curves.

3. Stormwater was identified as a major source of coliform in the lower Hillsborough River. Coliform concentrations violate existing water quality criteria.
4. Sediment oxygen demand resulting from solids deposition from Hillsborough Bay water intrusion, the extent of saline intrusion and upstream freshwater flow have a significant impact on dissolved oxygen concentrations in the lower Hillsborough River.
5. Certain stormwater control practices such as retention/detention ponds, ditches and drain trenches satisfy the State stormwater rule while implementation of storm sewers does not.

Receiving Water Study Conclusions

1. Dissolved oxygen concentration in the lower Hillsbourough River violate State criteria 20 to 60 percent of the time. DO levels are affected by the interaction of saltwater intrusion, freshwater discharges from the Hillsborough Reservoir and sediment oxygen demand. In general, dissolved oxygen concentrations are not seriously affected by stormwater runoff.

2. BOD concentrations in the lower Hillsborough River range from 1 to 4 mg/l and do not appear to increase significantly after storm events.
3. Coliform concentrations violate State criteria after storm events and occasionally during dry weather conditions. Coliform levels usually rise one or more orders of magnitude after a storm event.
4. Background levels of total suspended solids in the lower Hillsborough River range from 1 to 20 mg/l. Storm events cause concentrations to rise by 10 to 40 mg/l for a brief period. Metals present in the lower Hillsborough River are associated with suspended solids transported from urban areas.
5. Copper concentrations in the River violate State criteria after storm events. Lead concentrations also rise after storm events but do not violate criteria. Zinc levels decrease in wet weather, indicating the Hillsborugh Bay may be a source of this metal.
6. Nutrient concentrations are generally higher in the downstream reaches of the River. The intrusion of Hillsborough Bay water appears to be the primary source of these higher concentrations. Algal blooms were observed in various parts of the River during periods of low freshwater flows.

7. Thick muds comprise the River bottom in the downstream segments of the lower Hillsborough River. Sediment analyses indicate more silt, decaying material and high organic and nutrient content as one proceeds toward the River mouth.

8. The following management alternatives may improve water quality conditions in the lower Hillsborough River: 1) construction of a salt water barrier at the River mouth; 2) regulation of Reservoir releases; 3) dredging of bottom sediments; 4) disinfection of stormwater runoff prior to its release to the River; and 5) implementation of the Stormwater Management Plan.

9. With a salt water barrier at the River mouth future DO concentrations along the entire River without SOD would range from 5 to 7 mg/l.

10. A 30 percent reduction of stormwater loads to the River indicated an improvement of 0.2 to 0.5 mg/l in DO concentrations along the River.

Recommendations

A Stormwater Management Plan (SMP) has been developed for the City of Tampa to provide an integrated approach to solving the quantity and quality problems associated with stormwater that incorporates the relevant stormwater and receiving water environmental regulations. Based on the results of this Plan, several recommendations are made to the City. These include the following:

1. The implementation of the SMP should be expedited. The detailed planning phase of the SMP should replace the City's current Master Drainage Planning efforts. This will enable the City's Stormwater Division to proceed operations with a perspective on the overall stormwater problem in the City.
2. Stormwater controls that should be used in the City include retention/detention ponds, ditches, drain trenches and storm sewers. The former three controls will reduce both quantity and quality impacts on receiving waters of the lower Hillsborough River. The latter control will reduce flooding while generally increasing the pollutant load to the receiving waters.
3. To achieve maximal compliance to the State's Stormwater Rule, stormwater controls that alleviate quality impacts should be favored over controls that do not.
4. Detailed evaluations of economic and technical feasibility should be conducted to determine practicality of controls for water quality improvement. This will demonstrate to the State, through an assessment of alternatives, whether or not these controls should be implemented.

The water quality assessment of the lower Hillsborough River has indicated that a variety of factors affect the water quality conditions in this estuary. This assessment has led to the following recommendations for water quality improvements:

1. Construction of a salt water barrier to prevent the transport of BOD, organic solids and nutrients from Hillsborough Bay to the River should be considered. Further engineering analysis is necessary to investigate the economic feasibility and environmental and recreational impacts of this recommendation.
2. Dredging of the organic sediments in the River bottom to remove a significant oxygen demand in the downstream reaches of the lower Hillsborough River should be considered. Further investigation regarding the longitudinal and vertical extent of dredging must be performed to determine the feasibility of dredging. The frequency of dredging and the method of disposal of the dredged spoil should be investigated as well.
3. Future regulation of Reservoir release should be studied to ascertain the feasibility of a continual release from the Reservoir in order to improve the lower Hillsborough River water quality.
4. Implementation of the Stormwater Management Plan will improve the water quality in the lower Hillsborough River. Therefore, it is recommended that the City incorporates this Stormwater Management Plan into its future drainage system design and operation.

CHAPTER 2

INTRODUCTION

Background

The City of Tampa, Florida is experiencing rapid growth and urbanization, and associated flooding problems. Pollution of the lower Hillsborough River estuary, which runs through the center of the City, is partially attributable to stormwater runoff. A program designed to effectively address flooding and pollution concerns in the lower Hillsborough River Basin must incorporate State of Florida Department of Environmental Regulation criteria in stormwater permitting and water quality.

The implications of such an integrated approach to stormwater quantity and quality issues consistent with regulation have not been investigated. For this reason, the City of Tampa has participated in the Nationwide Urban Runoff Program (NURP). NURP is a program designed to determine the characteristics of stormwater runoff and the extent of runoff quantity and quality problems on a nationwide basis. Measurements and analyses were conducted at twenty-eight cities, of which Tampa was one.

The Tampa NURP study consisted of three phases as follows:

I. Assessment of stormwater loads and their receiving water impacts based on data collected prior to the NURP study.

II. Data collection and analysis of rainfall, stormwater runoff, and stormwater controls.

III. Development of a Stormwater Management Plan for the City of Tampa.

A related study on the impact of stormwater on the lower Hillsborough River was sponsored by the City of Tampa(1).

Report Scope and Organization

In this Summary report, Phases II and III and the Receiving Water Study are summarized and major findings of these efforts are identified and presented. The responsibilities and references for the detailed reports on individual Tasks are listed in Table 2-1.

A considerable amount of data was collected, analyzed and reported on during Phases II and III and the Receiving Water Study. This.final report is intended to demonstrate the critical aspects of these efforts and provide an integrated overview of the Tampa NURP study. Chapter 3 summarizes stormwater runoff measurements conducted during Phase II. Chapter 4 presents the stormwater runoff analysis. Lower Hillsborough River sampling and data are presented in Chapter 5 and modeling of the lower Hillsborough River is presented in Chapter 6. The stormwater management plan is presented in Chapter 7 while the supplements to the plan are presented in Chapter 8.

TABLE 2-1. SUMMARY OF TAMPA NURP PHASES II AND III TASKS

Task No.	Task name	Contractor	Reference
	Receiving Water Study	M&E	(1)
II.1	Procure and Install Gaging & Sampling Equipment	P-S[1]	(2)
II.2	Rainfall Quantity Analysis	M&E[2]	(3)
II.3	Rainfall Quality Analysis	P-S	(4)
II.4	Runoff Characterization - Basin Selection and Equipment Design	P-S	(5)
II.5	Runoff Characterization - Sampling	P-S	(6, 7)
II.6	Runoff Characterization - Analysis	M&E	(8)
II.7	Control Testing - Basin Selection and Equipment Design	P-S	(9)
II.8	Control Testing - Sampling	P-S	(10, 7)
II.9	Control Testing - Analysis	M&E	(8)
II.10	Projection of Study Area-wide Loadings	M&E	(8)
II.11	Monitor Other NURP Projects	M&E	(8)
II.12	Quality Assurance Program	P-S	(11)
III.1	Development of Stormwater Management Plan	M&E	(12)
III.2	Drainage Ordinances	P-S	(13)
III.3	Design Criteria	P-S	(14)
III.4	Drainage System Design	M&E	(12)
III.5	Operations and Maintenance	P-S	(15)
III.6	Long-Term Monitoring	M&E and P-S	(12, 16)

1. Priede-Sedgwick, Inc.
2. Metcalf & Eddy, Inc.

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5. Priede-Sedgwick, Inc., Runoff Characterization and Watershed Selection, Task II.4, Tampa Nationwide Urban Runoff Program Study, Phase II, December, 1982.

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11. Priede-Sedgwick, Inc., Quality Assurance/Quality Control Procedures Manual, Task II.12, Tampa Nationwide Urban Runoff Program Study, Phase II, December, 1982.

12. Metcalf & Eddy, Inc., Stormwater Management Plan, October, 1983.

13. Priede-Sedgwick, Inc., Drainage Ordinances, Task III.2, Tampa Nationwide Urban Runoff Program Study, Phase III, September, 1983.

14. Priede-Sedgwick, Inc., Design Criteria, Task III.3, Tampa Nationwide Urban Runoff Program Study, Phase III, September, 1983.

15. Priede-Sedgwick, Inc., Operations and Maintenance, Task III.5, Tampa Nationwide Urban Runoff Program Study, Phase III, September, 1983.

16. Priede-Sedgwick, Inc., Long Term Monitoring, Task III.6, Tampa Nationwide Urban Runoff Program Study, Phase III, September, 1983.

CHAPTER 3

STORMWATER RAINFALL/RUNOFF MEASUREMENTS

General

A major purpose of the Tampa NURP study was to collect sufficient data on rainfall and stormwater runoff to allow detailed evaluation of its impact. Specific goals of the field program were threefold:

1. Monitor rainfall at four sites to determine areal and temporal variations in rainfall quantity and quality.
2. Select five basins representative of land uses throughout the City, determine significant physical features, and monitor basin runoff for quantity and quality.
3. Locate appropriate test sites for four types of selected stormwater management practices, and monitor the quantity/quality responses of these practices to stormwater loading.

In this Chapter, the structure and results of the field program are described. This includes a description of the site locations, the monitoring equipment and the quality assurance/quality control procedures used. Then rainfall quantity and quality data are described. Finally basin selection for both runoff characterization and control testing is outlined, and the quantity and quality data collected in these basins are presented.

Field Measurement Procedures

Site Locations. The locations of the five runoff characterization basins and the twelve stormwater management practice test sites are shown in Figure 3-1. A number of the basins and test site stations are hydraulically connected, as suggested by their proximity.

The four precipitation quantity and quality monitoring sites are shown in Figure 3-2. This Figure shows the cumulative area of the basin(s) associated with each individual gage. The Horizon Park gage serves four individual basins including Wilder Ditch, North Jesuit High School, and North and South Ponds, which are all adjacent to each other. Additionally, the four original gages in the City monitoring network (A-D) and the NOAA Tampa International Airport (TIA) gage are shown in the Figure.

Monitoring Equipment and Facilities. The data collection activities which were conducted for the Tampa NURP Study included the following specific tasks:

1. Precipitation intensity, duration and cumulative amount measurement.
2. Composite and discrete sample collection.
3. Stage and/or flow measurements in control sections.
4. Water level measurements in North and South Ponds.

Primary factors considered in equipment selection were:

1. Total package capability for present and potential future data collection activities,
2. Documented unit reliability,

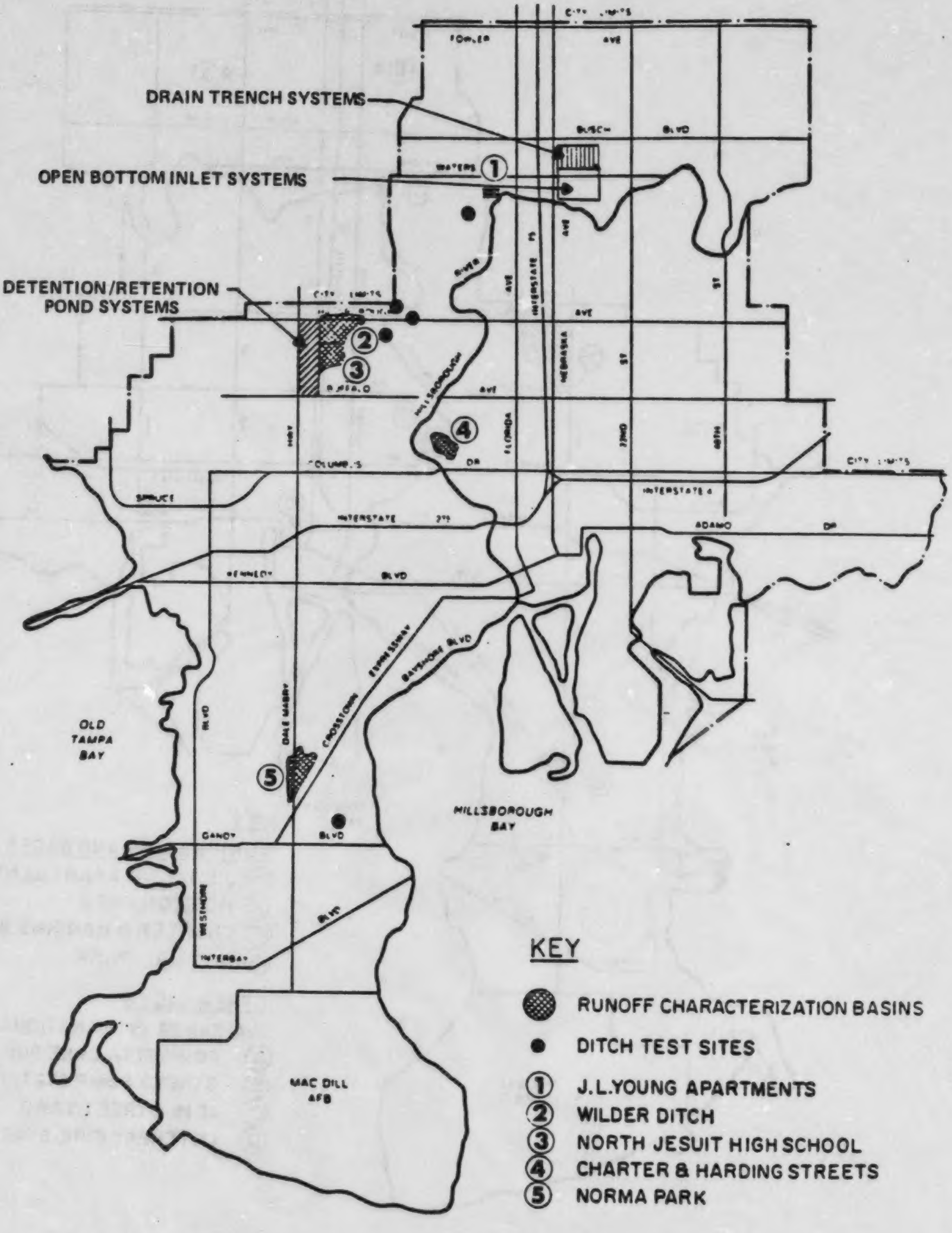
DRAIN TRENCH SYSTEMS
OPEN BOTTOM INLET SYSTEMS
DETENTION/RETENTION POND SYSTEMS
CITY LIMITS
FOWLER AVE
BUSCH BLVD
WATERS
RIVER
HILLSBOROUGH
AVE
INTERSTATE 75
NEBRASKA AVE
FLORIDA AVE
22ND ST
40TH ST
56 ST
HWY
BUFFALO AVE
COLUMBUS DR
SPRUCE
INTERSTATE 275
INTERSTATE 4
ADAMO DR
KENNEDY BLVD
BAYSHORE BLVD
CROSSTOWN EXPRESSWAY
DALE MABRY
BLVD
OLD TAMPA BAY
HILLSBOROUGH BAY
GANDY BLVD
WESTSHORE
INTERBAY
MAC DILL AFB
KEY
RUNOFF CHARACTERIZATION BASINS
DITCH TEST SITES
① J.L.YOUNG APARTMENTS
② WILDER DITCH
③ NORTH JESUIT HIGH SCHOOL
④ CHARTER & HARDING STREETS
⑤ NORMA PARK

FIG. 3-1 RUNOFF CHARACTERIZATION BASINS AND STORMWATER CONTROL TEST SITES

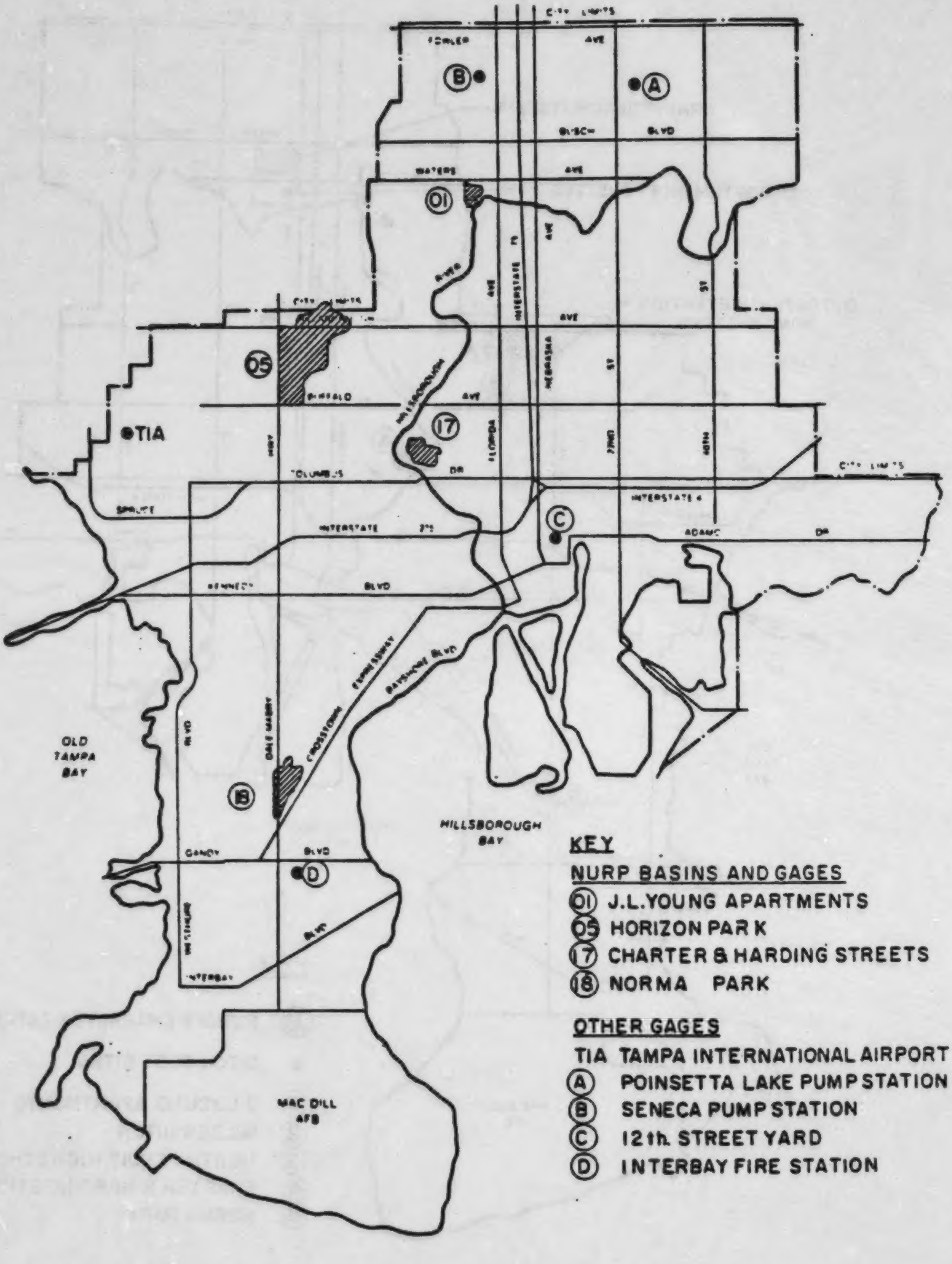
CITY LIMITS
FOWLER AVE
B
A
BUSCH BLVD
WATERS AVE
01
RIVER
AVE
INTERSTATE 75
AVE
ST
CITY LIMITS
AVE
05
HILLSBOROUGH
BUFFALO AVE
NEBRASKA
ST
TIA
17
HWY
FLORIDA
22ND
40TH
CITY LIMITS
COLUMBUS DR
SPRUCE
INTERSTATE 4
C
INTERSTATE 275
ADAMO DR
KENNEDY BLVD
CROSSTOWN EXPRESSWAY
BAYSHORE BLVD
OLD TAMPA BAY
BLVD
DALE MABRY
18
HILLSBOROUGH BAY
GANDY BLVD
D
WESTSHORE
BLVD
INTERBAY
MAC DILL AFB
KEY
NURP BASINS AND GAGES
01 J.L.YOUNG APARTMENTS
05 HORIZON PARK
17 CHARTER & HARDING STREETS
18 NORMA PARK
OTHER GAGES
TIA TAMPA INTERNATIONAL AIRPORT
A POINSETTA LAKE PUMP STATION
B SENECA PUMP STATION
C 12th. STREET YARD
D INTERBAY FIRE STATION

FIG. 3-2 PRECIPITATION GAGE LOCATIONS

3. Availability of field servicing by authorized or factory-trained technicians,

4. Unit costs submitted in bids,

5. Interchangeability of components.

Evaluation of the above resulted in the following manufacturers being selected:

Flow Measurement and Automated Sampling	Sigmamotor
Precipitation Measurement	Belfort
Water Level	Stevens

Instrument shelters housed all field equipment. These shelters were designed to prevent tampering, vandalism, and theft. The water level recorders in the ponds required the construction of docks to avoid the need for a boat.

Six groundwater monitoring wells were designed by Priede-Sedgwick, Inc. to monitor fluctuations in water table levels during the control testing period. All wells were installed by Pittsburg Testing Laboratory, Inc. of Tampa. Weekly field readings and test data summation were accomplished by City forces for the period May 1982 through February 1983.

The high and low water levels in the ponds and weir bays as well as the weir lengths were designed for each of the sites based on the flow regime. The individual weirs could pass substantially higher flows at higher stages in order to prevent flooding.

Control section decisions for other basins were based on analysis of the hydraulics of the storm sewer system. Two of the sites, J.L. Young Apartments and Charter & Harding Streets, used Palmer-Bowlus flumes for control sections while Norma Park utilized a combination of weir and open channel.

The four rain gages installed were each operated for about nine months. The gages were located to obtain measurements near the basins where detailed monitoring of runoff was to be conducted. The gages had devices which collected samples for rainfall quality analysis.

Quality Assurance/Quality Control. In order to assure the value of the field data, quality assurance/quality control (QA/QC) procedures for data collection and analysis were defined. These procedures were based on NURP QA/QC procedures and the standard QA/QC program utilized by Priede-Sedgwick, Inc., the consultant that conducted the field program. The City has obtained the approval of the USEPA NURP Staff for these procedures.

QA/QC procedures for field activities consisted of routine instrument service; field water quality analysis; and proper sample collection, identificaiton, preservation and handling. Laboratory QA/QC practices included laboratory certification, methods validation studies, audit sampling programs, analytical precision and accuracy maintenance, and continuous laboratory reporting.

Precipitation Quantity and Quality

Precipitation quantity data were collected to quantify the amount of rainfall associated with runoff characterization events in the monitored basins and to assist in evaluating whether rainfalls occurring during the study period were within normal ranges. Precipitation quality data were utilized in evaluation of contribution of various constituents from rainfall. The precipitation stations were operated continuously from their installation until termination of the field data collection activities. The gages recorded rainfall intensities, depths and durations for approximately nine months between May 1982 and February 1983.

Storm events at each of the four NURP Stations were analyzed using the rainfall intensity-duration-frequency curves developed during Task II.2 (1) to determine the distribution of storms between the identified recurrence intervals. This distribution is presented in Table 3-1.

Table 3-2 presents cumulative monthly information of three kinds: historical mean values, monthly observed volumes for the TIA gage, and the mean monthly volume for the Tampa NURP study precipitation gage network. The ratio of observed to historical mean values varies from 0.55 to 1.78 and the overall value for the study period is 1.26.

Rainfall samples for both dryfall and wetfall contaminants were collected and analyzed for an array of parameters including BOD_5, COD, TSS, pH, and several nutrients and metals.

TABLE 3-1. STORM DISTRIBUTION BY RECURRENCE INTERVAL FOR EACH BASIN

	J.L. Young Apartments	Horizon Park	Charter & Harding	Norma Park
Storm Event Total	56	49	27	51
Recurrence Interval, Months				
0 - 1 month	42	36	23	36
1 - 3 months	3	4	2	6
3 - 6 months	8	5	1	7
6 - 12 months	1	1	1	0
> 12	2	3	0	2

TABLE 3-2
PRECIPITATION VARIABILITY

	TIA GAGE(1)		NURP GAGES	NURP/TIA
	Historical Mean (in.)	Study Period (in.)	Network Mean(2) (in.)	Volumetric Ratio(3)
1982				
June	5.28	8.84	9.61	1.09
July	7.14	11.30	10.86	0.96
August	8.46	7.23	9.00	1.24
September	6.10	10.86	12.27	1.13
October	2.01	2.17	2.68	1.24
November	1.56	0.85	0.80	0.94
December	2.26	1.29	1.01	0.78
1983				
January	2.05	1.25	1.68	1.34
STUDY PERIOD	34.86	43.79	47.91	1.09

1. Tampa International Airport, NOAA Class A Gage No. 099788
2. Statistical mean of 4 NURP gages
3. Ratio = (NURP Mean Monthly Volume)/(TIA Monthly Volume)

Measurements were compared to assess areal variability of rainfall quality. Both areal and temporal variability in bulk precipitation samples were found to be significant and selected representative variabilities are summarized in Table 3-3.

Runoff Characterization-Basin Selection

Initial site selection criteria included the following:

1. Uniformity of land use and development pattern.
2. Representative land uses in the City.
3. Relative assurance of land use stability.
4. Watershed area of 10-200 acres.

Final selection entailed determining that the site can provide the information desired. Familiarity with the storm sewer system was necessary, and knowledge regarding inflow or outflow between the measuring points and the point of data collection was essential. The selected basins are described below.

J.L. Young Apartments. This 8.8 acre basin is a high-density residential complex located in north-central Tampa. Due to the high percentage of impervious surface and the extensive storm sewer system in the basin, its hydraulic response to rainfall tends to be rapid. All flows from this basin meet in a flat reinforced concrete junction box located at the approximate midpoint of the southern boundary of the basin. From this point, the stormwater flows south to the Hillsborough River via a 36 inch diameter outfall.

TABLE 3-3. PRECIPITATION QUALITY

Parameter	Unit	Statistic	J.L. Young Apartments	Horizon Park	Charter & Harding	Norma Park	All Stations
COD	mg/l	min	4	5	<1	3	<1
		mean	12	17	17	20	17
		max	23	49	38	43	49
Total-P	mg/l	min	<0.02	0.01	0.026	0.036	0.01
		mean	0.16	0.10	0.13	0.23	0.16
		max	0.62	0.21	0.50	0.70	0.70
Nitrate	mg/l	min	0.13	0.11	0.08	0.135	0.08
		mean	0.19	0.24	0.24	0.30	0.24
		max	0.34	0.58	0.50	0.70	0.70
TKN	mg/l	min	0.10	<0.13	<0.1	<0.10	<0.10
		mean	0.34	0.38	0.55	0.54	0.45
		max	1.31	0.85	1.44	2.22	2.22
Organic-N	mg/l	min	<0.10	<0.10	<0.10	<0.01	<0.01
		mean	0.34	0.34	0.53	0.48	0.42
		max	1.31	0.56	1.44	2.22	2.22
Total-N	mg/l	min	0.13	0.37	<0.1	0.22	<0.10
		mean	0.51	1.23	0.77	0.82	0.83
		max	1.48	4.78	1.72	2.36	4.78
pH	--	min	5.0	5.30	4.81	4.95	4.81
		mean	5.46	5.65	5.40	6.01	5.63
		max	6.70	6.00	6.50	7.05	7.05
Cadmium	mg/l	min	0.2	0.3	0.5	0.3	0.20
		mean	0.55	0.7	1.5	1.0	0.94
		max	0.9	1.4	3.6	2.0	3.60
Copper	mg/l	min	25	6.6	16	13.3	6.6
		mean	53	32	50	24.5	39.9
		max	98	86	103	50	103
Lead	mg/l	min	3.1	2.3	2.0	2	2
		mean	6.7	7.3	3.8	5.6	5.85
		max	14.2	10.8	5.3	10	14.2
Zinc	mg/l	min	65	30	60	85	30
		mean	189	92	183	119	145.8
		max	400	195	360	180	400

Monitoring efforts required the installation of a flow meter interfaced to an automatic sampler. The Palmer-Bowlus flume control section was located in the 36 inch outlet pipe. The flow meter, automatic sampler, and rain gage were located in an instrument shelter approximately 75 feet due west of the monitoring point.

Wilder Ditch. This 193.9 acre basin contains predominantly residential land with some commercial and institutional areas. The monitoring site for the basin was located on the western side of the drainage area where the ditch system flows into the North Pond. The control section was a ten-foot long, sharp-crested weir immediately downstream from a double, 3x10 foot box culvert. An instrument shelter, housing a sampler and flow meter was located approximately 15 feet north of the control section. Precipitation measurements were made with the rain gage in Horizon Park at the South Pond Discharge.

North Jesuit High School Basin. This 29.5 acre basin contains a mixture of land uses including low density residential, institutional, and commercial. The basin drains under Himes Avenue into the South Pond through a 38x60 inch elliptical pipe. Under normal water level conditions in the pond, the pipe is partially submerged.

The control section, a 6.0 foot wide sharp-crested weir, was located on the western side of the drainge area where the storm sewer system flows into South Pond. Both a flow meter and an automatic sampler were located in an instrument shelter approximately 15 feet east of the control section. Precipitation measurements were obtained with the rain gage located at the South Pond Discharge in Horizon Park approximately 700 feet north of the site.

Charter & Harding Streets. This 46.1 acre basin contains primarily residential land uses, with the exception of a small neighborhood park, and drains south to the Hillsborough River. The stormwater system in this basin consists of a combination of open ditches and closed conduits which meet in a large junction box at the intersection of Charter & Harding Streets. Monitoring activities were conducted at the basin outlet prior to direct discharge into the Hillsborough River.

The control section was a 36 inch Palmer-Bowlus flume located in the discharge pipe in Charter Street just west of the junction box. Instrumentation included an automatic flow meter interfaced to an automatic sampler. A rain gage was installed at the southwest corner of Charter Street and Perry Avenue, two blocks west inside the sanitary sewer pump station compound. The instrument shelter housing the sampler and flow meter was located approximately 35 feet southeast of the control section.

Norma Park. This basin contains two commercial shopping centers and highway commercial areas draining into the Norma Park ditch system. It was selected as representative of high density commercial development in Tampa. Land use in the basin is 91 percent commercial.

The control section chosen was an 8-foot sharp-crested weir without end contractions embedded in the existing rip-rapped trapezodial channel section. Flow monitoring and sampling were conducted at the basin outlet located on the western side of the drainage area approximately 17 feet west of the Dale Mabry Highway right-of-way. Instrumentation included a flow meter interfaced with an automatic sampler which were enclosed in an instrument shelter located approximately 12 feet northwest of the control section. A rain gage was located on the shelter.

Basin Characteristics Summary. Detailed site specific investigations formed the basis for describing the five selected basins. Many of the important characteristics for these five urban runoff characterization basins are summarized in Table 3-4.

Runoff Quantity and Quality

Monitoring activities began upon the completion of basin selection and equipment acquisition and installation. Hydraulic characteristics were monitored continuously throughout the study period. Composite and discrete samples were collected at each station and were analyzed for an array of standard water quality parameters.

Quantity. The continuous hydraulic records on the flow meter strip charts at the five runoff characterization basins were reviewed, reduced to a series of flow characteristics and combined with the recorded precipitation characteristics.

TABLE 3-4. CHARACTERISTICS OF MONITORING BASINS

J.L. Young Apartments		Wilder Ditch	North Jesuit High School		Charter & Harding Street	Normal Park Ditch
Area (acres)		8.8	193.9	29.5	42.2	46.6
Maximum Length (ft.)		460	5300	1800	1720	2200
Maximum Width (ft.)		830	1120	1570	1480	1530
Length to Width Ratio		0.6	4.7	1.2	1.2	1.5
Minimum Elevation (ft., MSL)		21	29.3	28.7	15	15
Maximum Elevation (ft., MSL)		36	46.3	35	22.6	16.5
Normal Elevation Range (ft., MSL)		27–30	35–42	30–32	18–21	15–16
Predominant Ground Slope Direction		South	West	West	South	South
Representative Slope (ft/ft)		0.023	0.0029	0.0027	0.003	0.001
Land Use						
Low-density Residential	acres	0	105.7	14.1	37.6	0
	(% of basin)	(0)	(54.5)	(47.8)	(89.1)	(0)
Medium-Density Residential	(acres)	0	0	0	0	4.3
	(% of basin)	(0)	(0)	(0)	(0)	(9.3)
High-density Residential	(acres)	8.8	0	0	0	0
	(% of basin)	(100)	(0)	(0)	(0)	(0)
Commercial	(acres)	0	48.1	0	0	42.3
	(% of basin)	(0)	(24.8)	(0)	(0)	(90.7)
Institutional	(acres)	0	14.4	15.4	0	0
	(% of basin)	(0)	(7.4)	(52.2)	(0)	(0)
Open	(acres)	0	25.8	0	4.6	0
	(% of basin)	(0)	(13.3)	(0)	(10.9)	(0)
Total	(acres)	8.8	193.9	29.5	42.2	46.6
	(% of basin)	(100)	(100)	(100)	(100)	(100)

These characteristics include storm number, date, time the event began, duration of the event, total rainfall volume, mean intensity, runoff volume, peak flow rate, time to peak and mean flow rate.

These ten characteristics have been summarized for each of the five runoff characterization stations. The extent of areal and temporal variability in these data is summarized in Table 3-5 for selected rainfall and runoff characteristics.

Quality. The field program called for the collection of water quality samples at each of the five stations in the following manner: twelve composite samples generated on a flow-weighted basis for basin characterization, and three discrete series of eight individual samples to describe temporal variations in within-storm water quality. The distribution of composite and discrete samples among the five stations is summarized in Table 3-6.

The composite samples collected at the characterization basins showed both temporal and areal variability. Table 3-7 summarizes the variability for the composite samples of some of the representative parameters measured.

Ambient water quality measurements were conducted during the first half of the field program. Twenty-one ambient measurements were taken at the five stations. These data were analyzed, and Table 3-8 presents their characteristics.

TABLE 3-5
RAINFALL AND RUNOFF EVENT CHARACTERISTICS

	J.L. Young Apartments	Wilder Ditch	North Jesuit	Charter & Harding	Norma Park
Number of Events	57	77	77	57	51
Event Duration (hours)	0.25-22.50	0.20-29.67	0.20-29.67	0.04-5.65	0.13-7.42
Total Rainfall (inches)	0.04-4.10	0.03-4.88	0.03-4.99	0.04-5.65	0.05-4.27
Mean Flow Rate (MGD)	0.00-2.37	0.00-14.13	0.00-16.48	0.00-7.90	0.07-3.45
Peak Flow Rate (MGD)	0.00-14.09	0.00-49.20	0.00-57.15	0.00-37.93	0.30-20.0
Time to Peak (hours)	0.00-18.58	0.00-18.75	0.00-20.03	0.00-17.43	0.17-4.63
Runoff Volume	0.00-1.35	0.00-28.95	0.00-16.47	0.00-5.25	0.05-1.65

TABLE 3-6. BASIN SAMPLE DISTRIBUTION

	J.L. Young Apartments	Wilder Ditch	North Jesuit	Charter & Harding	Norma Park	All Stations
Samples:						
Composite	12	15	15	12	12	66
Discrete	21	24	20	22	24	111

TABLE 3-7
SUMMARY OF COMPOSITE DATA

Parameter	Statistic	J.L. Young Apartments	Wilder Ditch	North Jesuit	Charter & Harding	Norma Park
BOD_5(1)	min	2	<2	<2	<2	<2
	mean	15	14	14	12	11
	max	28	29	26	31	20
COD	min	7	20	<5	20	24
	mean	64	48	44	55	40
	max	141	86	110	161	145
TSS	min	8	10	<2	<2	2
	mean	49	34	68	30	19
	max	122	90	314	140	54
Total-P	min	0.10	0.07	0.03	0.05	<0.05
	mean	0.33	0.23	0.19	0.40	0.15
	max	0.76	0.49	0.31	2.14	0.26
Nitrate	min	0.08	0.07	0.07	0.15	0.20
	mean	0.29	0.43	0.35	0.62	0.34
	max	0.64	0.97	0.65	1.58	0.82
Nitrite	min	<0.002	0.003	<0.005	<0.001	<0.005
	mean	0.015	0.021	0.020	0.024	0.02
	max	0.032	0.048	0.070	0.12	2.32
Ammonia	min	<0.01	<0.03	<0.01	<0.10	<0.03
	mean	0.17	0.23	0.23	0.17	0.19
	max	1.16	0.68	0.79	0.55	1.74
Organic-N	min	0.17	0.13	0.53	0.37	<0.10
	mean	1.12	0.91	1.18	1.54	0.65
	max	2.10	1.60	2.87	4.03	2.72
Total-N	min	0.40	0.58	0.82	0.53	0.74
	mean	1.57	1.55	1.73	1.01	1.13
	max	2.66	2.40	4.00	4.96	4.00

1. All parameters expressed in mg/l

TABLE 3-8
AMBIENT WATER QUALITY CHARACTERISTICS

Parameter	Statistic	J.L. Young Apartments	Wilder Ditch	North Jesuit	Charter & Harding	Norma Park
Flow (MGD)	min	0.40	1.20	0.18	0.07	4.10
	max	4.30	>31	8.95	16.50	5.25
	range	3.90	NA(1)	8.77	16.4	1.15
	mean	1.65	3.51	2.51	5.73	4.68
Dissolved Oxygen (mg/l)	min	1.0	1.3	0.3	2.4	2.9
	max	7.3	3.3	5.7	6.1	6.2
	range	6.3	2.0	5.4	3.7	3.3
	mean	5.0	2.4	2.9	4.4	3.9
Specific Conductivity (umhos/cm)	min	217	385	314	329	407
	max	234	430	415	329	440
	range	17	45	101	0	33
	mean	226	408	364	329	423
pH (standard units)	min	6.73	6.88	6.62	6.87	6.86
	max	7.32	7.31	7.32	7.27	7.16
	range	0.59	0.43	0.70	0.40	0.30
	mean	6.99	7.10	6.98	7.04	6.98
Water Temperature (deg C)	min	20.3	23.7	20.2	20.7	24.3
	max	21.2	25.9	28.3	26.7	27.6
	range	0.9	2.2	8.1	6.0	3.3
	mean	20.7	25.1	24.1	24.2	26.3

1. Not available

Data collected for each sampled event was organized into three forms which included:

(1) a detailed event summary,

(2) a precipitation event analysis, and

(3) a discharge hydrograph for the sampled period.

Control Testing-Basin Selection

Four types of stormwater controls were selected for evaluation of their effectiveness in reducing stormwater quality and quantity loads. These included open bottom inlets, drain trenches, ditches/swales, and detention/retention ponds. Control selection was based on the availability of controls, current City priorities for control use, and suitability for monitoring. Ideally, the controls would be located adjacent to a runoff characterization basin.

The ability of open botttom inlets, drain trenches and ditches to attenuate stormwater hydraulic loads was evaluated, without testing water quality. Both the water quality and quantity characteristics of retention/detention ponds were tested. Such data allowed for determination of the effectiveness of the management practice to remove pollutant loads and/or attenuate hydraulic loads associated with stormwater runoff.

Open Bottom Inlets. The two open bottom inlet (OBI) test sites with three individual OBIs are located in the Waters Avenue Storm Sewer basin. This area is located north of the Hillsborough River and south of Busch Boulevard, between 13th and 18th Streets.

The first open bottom inlet test site (OB-1) is located on the northeast corner of Brooks and Bird Streets. The inlet is a standard open bottom inlet that discharges to a manhole in the main trunk of the storm sewer system that flows east in Bird Avenue. Contributing drainage area is estimated as 2.0 acres.

The second test site (OB-2A, OB-2B) contains the OBIs on the southeast corners of Bird and Marks Streets. Contributing drainage area to the OBI on the southeast corner is estimated to be 0.3 acres. Contributing drainage area to the OBI on the northeast corner is estimated at 2.1 acres. Both the OBIs have standard inlets that discharge to the same junction box.

Drain Trench Systems. The two drain trench test sites are located in the basin served by the recently constructed 9th Street Storm Sewer Extension. The first test site (DT-1) is located on the northeast corner of Fairbanks Street and 11th Street. The drain trench inlet box is a standard curb inlet connected to another inlet located on the northwest corner of Fairbanks and 11th Streets. Both inlets collect stormwater generated in areas north of the intersection that flow south in gutters to the inlets. Two drain trenches are connected to the inlet. One trench runs east from the box for 22 feet. The other runs north from the inlet box for 71 feet. The drainage area associated with this test site is 1.1 acres.

The second test site is located on the southeast corner of Fairbanks and 11th Streets. The three inlets are standard curb with one inlet located adjacent to the trench manhole. Another inlet is located northeast of the drain trench inlet on the southern edge of pavement of Fairbanks Street. A third inlet is located on the southwest corner of 11th and Fairbanks Streets. The drain trench is located in the eastern right-of-way of 11th Street, south of Fairbanks Street, and is 84 feet long. The drainage area associated with this test site is 2.1 acres.

Ditches/Swales. Ditch test sites are located primarily in the northern portion of the City around Hillsborough and Armenia with the exception of one site located in the southern section of the City just north of Gandy Boulevard. Ditch systems exist throughout Tampa.

The ditch test site locations and characteristics are listed in Table 3-9.

TABLE 3-9. DITCH TEST SITE LOCATIONS AND CHARACTERISTICS

Ditch	Slope (percent)	Length (ft)	Location
DS-1	0.46	169	Southern right-of-way of Powhattan Ave. between Habana and Tamoania Aves.
DS-2	0.42	214	Eastern right-of-way of Habana Ave. between Louisiana and Osborne Aves.
DS-3	0.75	362	Northern right-of-way of Comanche Ave. between Albany and Rome Aves.
DS-4	0.47	175	Southern right-of-way of Kirby St. between Newport Ave. and North Blvd.
DS_5	0.43	220	Western right-of-way of Datura Ave. between Alline and Beaumont Aves.

Retention/Detention Ponds. It was desirable that the ponds have a limited number of inflow points which are suitable for monitoring and that they are located immediately downstream of basins selected for urban runoff characterization. These considerations helped maintain a financially viable program.

The two detention/retention ponds selected are located in Horizon Park, a City-owned recreational facililty. The ponds are designated North and South Ponds. The North and South Jesuit Basins drain into the South Pond which, in turn, discharges into the North Pond. Wilder Ditch discharges directly to the North Pond which, in turn, discharges to a major ditch system along the Dale Mabry Highway, eventually flowing into Old Tampa Bay.

The South Pond is located in the southern one-third of the 126 acre Horizon Park area. This pond has a surface area of approximately 2.9 acres. The North Pond is substantially larger than the South Pond and is located in the central portion of the northern two-thirds of the park. The surface of the North Pond is approximately 9.1 acres. Monitoring was conducted at four stations while testing the detention/retention ponds, leaving the South Jesuit inflow unmonitored.

Control Testing Quantity and Quality

Two tests for quantity only were conducted at each of the ten test sites for open bottom inlets, drain/trenches and ditch systems to document the hydraulic extraction capacity of these controls after different periods of rainfall. Discrete sample series of quantity and quality for five storms at both ponds were collected and analyzed.

Open Bottom Inlets. The inflow/outflow characteristics and seasonal time of testing for each test at all three sites are summarized in Table 3-10.

Drain Trench Systems. The data for net performance of the two systems in each test, and the seasonal time of testing is summarized in Table 3-11 for each test site.

TABLE 3-10. OPEN BOTTOM INLET TEST SUMMARY

Test Site	Test Number	Season	FLOW RATES (cfm) Inflow	Outflow	Extraction
OB-1	1	Wet	1.37	1.32	0.05
	2	Dry	7.00	4.68	2.32
OB-2A	1	Wet	7.28	7.28	0
	2	Dry	2.17	2.17	0
OB-2B	1	Wet	5.80	5.68	0.12
	2	Dry	3.75	3.42	0.33

TABLE 3-11. DRAIN TRENCH TEST SUMMARY

Test Site	Test Number	Season	FLOW RATES (cfm) Inflow	Outflow	Extraction
DT-1	1	Wet	2.6	4.9	2.3
	2	Dry	6.0	21.2	15.2
DT-2	1	Wet	6.0	21.8	15.8
	2	Dry	3.4	9.7	6.2

Ditch Systems. Table 3-12 presents the reduced test data for each test as well as the gross and unit extraction rates for ditch systems. Unit rates, expressed in cubic feet per minute per foot of ditch (cfm/ft), facilitates comparison of ditches. In the case of stepped hydrograph loading, the table reports the values associated with the highest sustained volumetric extraction conditions.

TABLE 3-12. DITCH SYSTEMS TEST SUMMARY

Test Site	Test No.	Extracted Volume (cu. ft.)	Elapsed Time (min.)	Steady-State Average Peak of Extraction (cfm)	Extraction Rate Gross (cfm)	Extraction Rate Unit (cfm/LF)
DS-1	1	443	87.5	5.06	3.63	0.0198
	2	629	92	6.83	5.04	0.0298
DS-2	1	957	120	7.98	10.47	0.0431
	2	718	95	7.56	6.04	0.0249
DS-3	1	201	62	3.25	1.76	0.0049
	2	381	64	5.95	3.84	0.0106
DS-4	1	159	84	1.90	1.12	0.0064
	2	303	72	4.21	2.99	0.0171
DS-5	1	421	53	7.95	5.17	0.0235

Detention/Retention Ponds. Rainfall and the resultant stormwater runoff were measured for flows through the Horizon Park pond system for the period of May 1982 to February 1983 in order to define the inter-system hydraulic characteristics.

The hydraulic records on the strip charts were reduced to a series of flow characteristics and combined with the precipitation characteristics monitored at the Horizon Park precipitation gage. These characteristics include storm number, date, time the event began, event duration, total precipitation volume, mean intensity, runoff volume, peak flow rate, time to peak, and mean flow rate. These ten characteristics have been summarized for each station in the Horizon Park pond system. The stage of both ponds during these events is important due to the relationship between pond stage and volume. Pertinent water surface elevation information is available for the South and North Ponds at selected times during the discrete sampling events.

Discrete and composite water quality samples were collected at the four monitoring stations throughout the sampling period and these samples were analyzed by Interscience, Inc. of Tampa, Florida. In addition, several in-situ outfall, in-situ pond and bench tests were performed at the four stations.

Discrete series sample data for the two pond stations, South Pond Discharge and North Pond Discharge have been reviewed, tabulated and summarized as flow-weighted composite data in Table 3-13.

TABLE 3-13. FLOW-WEIGHTED COMPOSITE POND QUALITY DATA RANGES

Parameter	Units	South Pond Discharge Range	North Pond Discharge Range
BOD	mg/l	6-11	6-10
COD	mg/l	30-74	30-86
TSS	mg/l	2-11	4-15
Ortho-P	mg/l	0.01-0.08	0.02-0.06
Total-P	mg/l	0.05-0.10	0.07-0.10
Nitrate	mg/l	0.07-0.14	0.10-0.23
Nitrite	mg/l	0.00-0.02	0.01-0.02
TKN	mg/l	0.61-1.53	0.90-1.19
Ammonia	mg/l	0.03-0.51	0.10-1.01
Organic-N	mg/l	0.59-1.40	0.87-1.28
Total-N	mg/l	0.86-1.62	0.67-1.27
Cadmium	mg/l	0.7 -1.3	1.4 -9.9
Copper	mg/l	1.1 -5.7	2.6 -13.0
Lead	mg/l	3.6 -7.0	4.8 -32.0
Zinc	mg/l	22-67	17-34

Summary

In this chapter, the details of the extensive rainfall/runoff and control testing field program conducted for the City by Priede-Sedgwick, Inc. were described. These details indicate that the goals of the field program as listed in the first section of this chapter were satisfied.

REFERENCES

1. Metcalf & Eddy, Inc., Rainfall Quantity Analysis, Task II.2, City of Tampa, Tampa Nationwide Urban Runoff Program Phase II, February, 1983.

CHAPTER 4

STORMWATER RUNOFF ANALYSIS

General

This Chapter addresses the analysis of the rainfall-runoff data collected at the five monitoring basins as well as the data from the control testing, all of which are described in Chapter 3. These data were used to evaluate and characterize the following:

1. Rainfall quantity,
2. Runoff quantity and quality,
3. Control effectiveness, and
4. Load projections.

The rainfall quantity analysis was performed using standard statistical and hydrologic methods such as intensity-duration-frequency analysis. Within-storm behavior of runoff quantity and quality was analyzed with a stormwater management model. The behavior of storm averaged parameters was studied through a statistical analysis. Control effectiveness was estimated by analyzing removal rates in the test series. Load projections were made by determining the characteristics of the overall study area and routing selected annual rainfall histories through it.

Rainfall Quantity Analysis

The objective of the rainfall quantity analysis was to identify rainfall characteristics in Tampa; develop design storms; and select typical dry, average and wet years for use in load projections. Hourly precipitation data collected at the

Tampa International Airport Class A Weather Station were used for the analysis. After elimination of years with missing hourly data, twenty-two calendar years of hourly data (1949, 1951, 1959-1974, 1976-1979) were available and were used in the analysis.

Rainfall Characteristics in Tampa. The hourly precipitation records were converted into individual storm events. A storm event was defined as recorded rainfall preceded by or separated from other recorded rainfall by at least six hours. Storms with a total rainfall less than 0.10 inches were neglected since they do not contribute significant runoff. The storm events were categorized by four different rankings: chronology, maximum intensity, duration, and total rainfall.

Figure 4-1 presents the distribution of the 1,395 storm events with rainfall greater than 0.10 inch in terms of storm duration and peak intensity. These statistics show that the most common storms are of short duration and high frequency. In the summer months these take the form of short, intense thunderstorms that occur frequently and have the potential to wash off large pollutant loadings. Therefore, design storms with shorter durations and low return periods were developed for use in runoff analysis.

Design Storm Development. A range of design storms was developed with return periods of one, three, six, and twelve months and durations of one, two, three and six hours. The intensity-duration-frequency curves determined for the four

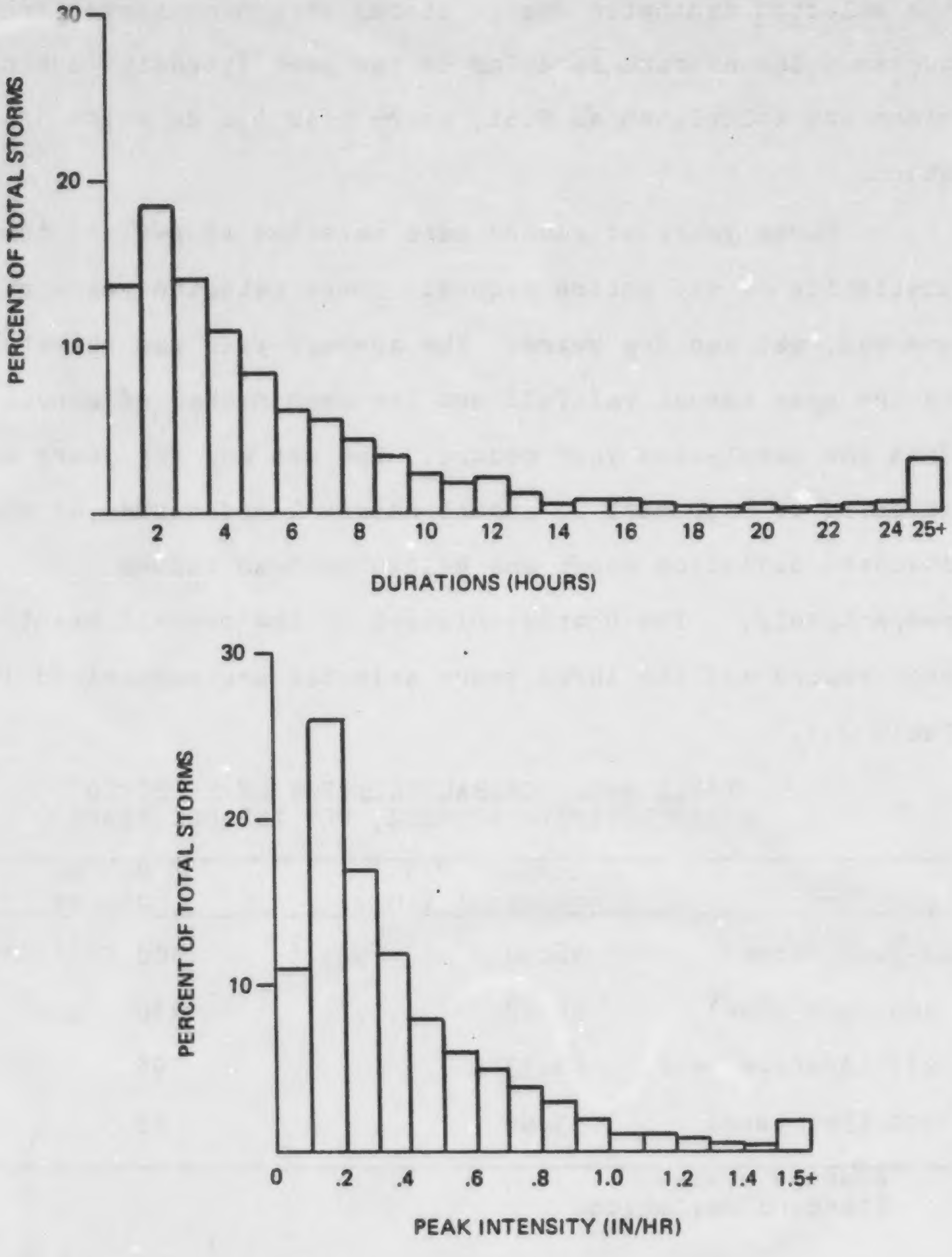

FIG. 4-1 DISTRIBUTION OF STORMS BY HOURLY DURATION AND PEAK INTENSITY

return periods are shown in Figure 4-2. Rainfall hyetographs for the selected synthetic design storms were constructed from these curves. The average location of the peak intensity during a storm was calculated as 0.5t, where t is the duration of the storm.

Three years of record were selected to reflect the statistics of the entire record. These selected years represent average, wet and dry years. The average year was selected based on the mean annual rainfall and the mean number of annual storms from the twenty-two year record. The wet and dry years were selected to represent an annual rainfall and number of storms one standard deviation above and below the mean values respectively. The characteristics of the overall twenty-two year record and the three years selected are summarized in Table 4-1.

TABLE 4-1. CHARACTERISTICS OF SELECTED REPRESENTATIVE AVERAGE, WET AND DRY YEARS

Year	Annual rainfall (in.)	No. of storms
22-year record	46.10(1) (11.53)(2)	100.5(1) (10.7)(2)
1969 (Wet year)	54.22	110
1971 (Average year)	46.33	95
1974 (Dry year)	33.90	78

1. Average value.
2. Standard deviation.

Multiple Event Storms. A multiple event storm includes two or more storm events occurring at least 6 hours or more after

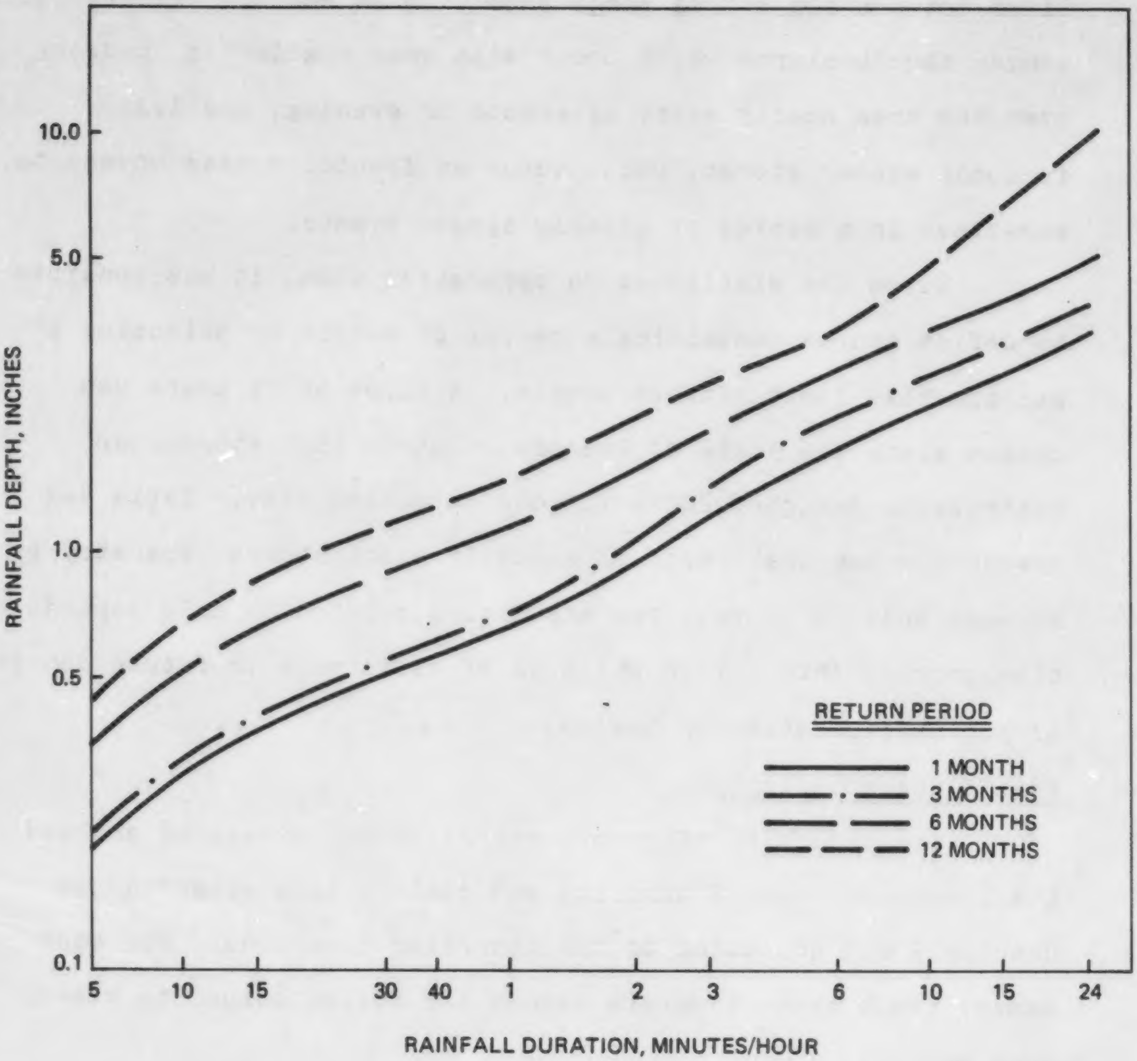

FIG. 4-2 RAINFALL INTENSITY-FREQUENCY-DURATION CURVES FROM TAMPA INTERNATIONAL AIRPORT RAINFALL RECORDS

the preceding event. Figure 4-3 presents the distribution of the separation time between storms. The most frequent separation times between two storms range from 7 to 24 hours. This reflects summer thunderstorms which occur with some regularity, passing over the area nearly every afternoon or evening, and less frequent winter storms, which occur as frontal system movements, sometimes in a series of closely spaced events.

Given the statistics on separation time, it was possible to define storms containing a series of events by selecting a maximum time limit between events. A value of 72 hours was chosen since the State of Florida requires that stormwater controls be designed for a 72 hour detention time. Table 4-2 summarizes the statistics of multiple event storms separated by no more than 72 hours. The statistics related to this separation time provide information which is of assistance in future design of stormwater detention basins.

Runoff Characterization

The objective of runoff characterization was to analyze the stormwater runoff quantity and quality data described in Chapter 3 and collected at the described locations. For each basin, about three discrete events and twelve composite events were sampled.

The discrete event stormwater sampling program was conducted to monitor within-storm variation of rainfall and stormwater quantity and quality for representative drainage basins in Tampa. Analysis of this within-storm variability was

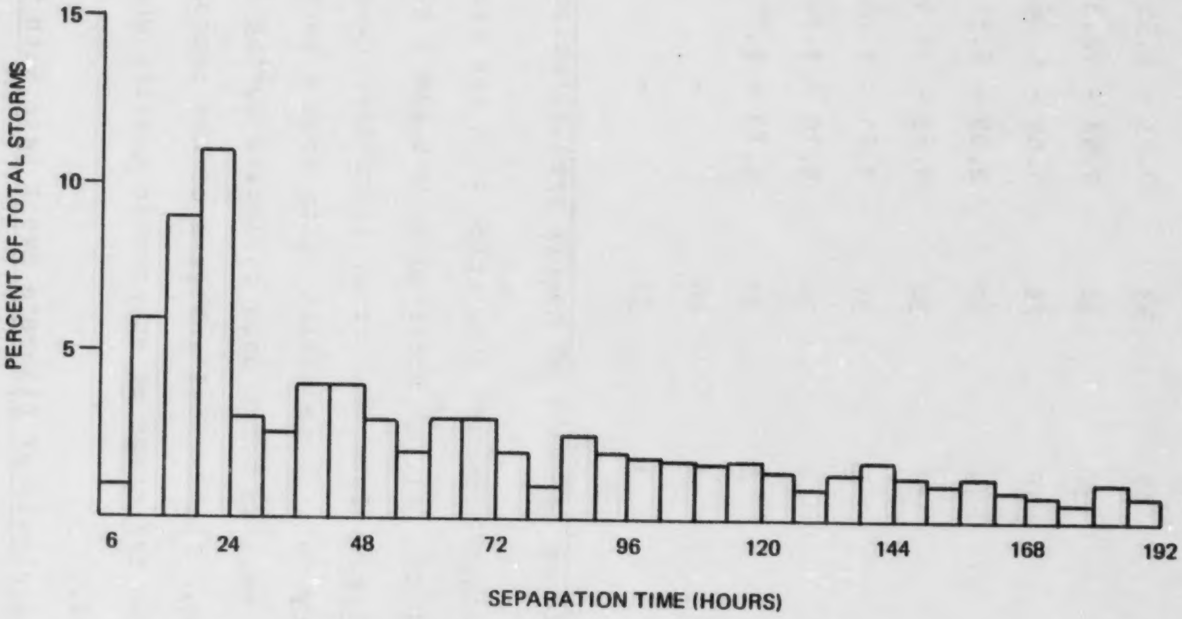

FIG. 4-3 DISTRIBUTION OF SEPARATION TIME BETWEEN STORMS IN TAMPA

TABLE 4-2. MULTIPLE EVENT STORM STATISTICS FOR A MAXIMUM SEPARATION TIME OF 72 HOURS

No. of storms in event	No. of events(1)	Average separation time (hrs)	Range of total rainfall (in)	Average total rainfall (in)
2	160	33	0.27 - 7.12	1.43
3	67	33	0.43 - 9.35	2.20
4	32	30	0.93 - 10.39	3.33
5	16	28	1.09 - 6.26	3.32
6	10	32	2.08 - 8.57	4.32
7	13	30	1.63 - 19.63	5.87
8	9	31	1.81 - 7.08	4.50
9	4	30	5.19 - 7.79	6.93
10	2	39	6.89 - 9.70	8.30
11	1	26	-	4.99
13	1	36	-	6.84

---

1. Based on 22 years of hourly precipitation records.

done through computer simulation with the Storm Water Management Model (SWMM) (1), as modified by Metcalf & Eddy, Inc. Because of the highly variable nature of stormwater quality, it is sometimes necessary to analyze quality data from a large number of storm events, such as those data collected during the composite event sampling. Statistical analysis of the composite data was used to determine the range of stormwater quality characteristics expected.

Analysis of Discrete Event Data with SWMM. Hydraulic simulation of discretely sampled events was performed for each of the five monitored drainage basins. This entailed dividing each basin into subcatchments based on existing drainage patterns. The total area and shape, percent impervious area and ground slope of each subcatchment was specified based on data presented in Chapter 3. Additional runoff factors such as surface resistance coefficients, surface retention depths, infiltration rates and impervious area with no storage were typically set to SWMM default values.

At the beginning of the rainfall event, the portion of the rain which becomes runoff proceeds to the collection system to be transported downstream. The transport process requires that a physical description of the collection system be specified, including the size, slope and roughness coefficients for storm sewers and ditches. Details of the drainage system were also determined from Chapter 3 data. Additional input included any base flow or estimated groundwater infiltration entering the collection system during a particular storm event.

Given the physical characteristics of the basins, the SWMM produces an outflow hydrograph at the most downstream point in the basin. In Figure 4-4, a typical simulated hydrograph for an event modeled with the SWMM is compared to a measured hydrograph. In general, the outflow hydrographs computed by SWMM compared favorably with the measured hydrographs.

Through the SWMM hydraulic calibration and verification process, the following factors were determined to have an important influence upon runoff generation:

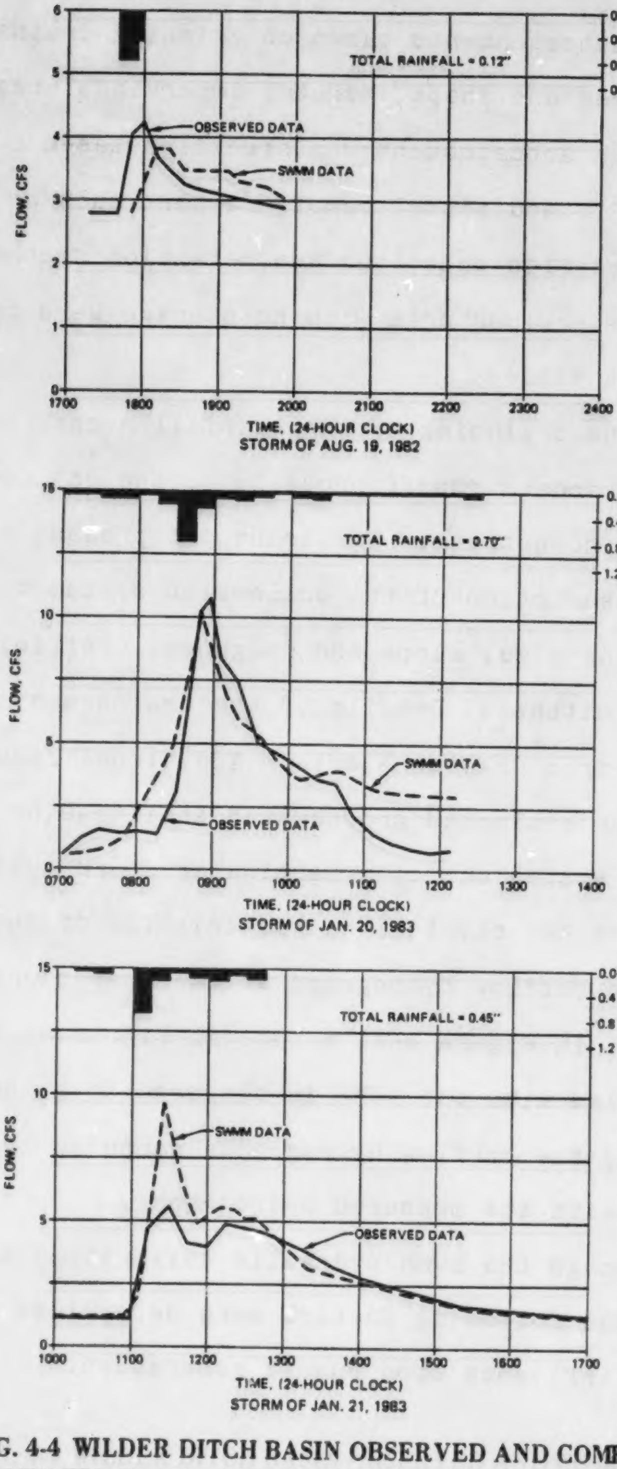

FIG. 4-4 WILDER DITCH BASIN OBSERVED AND COMPUTED STORMWATER FLOW DATA, CALIBRATION AND VERIFICATION

1. Rainfall,
2. Infiltration,
3. Basin and transport characteristics,
4. Ditch systems,
5. In-system storage.

The calibration and verification runs were performed with a minimum of changes in input parameters. Only some of the parameters listed above were modified, and these modifications were only required in special instances.

After SWMM was hydraulically calibrated to describe the measured outflows in each basin, the computed outflow concentrations were compared with the quality data obtained during the discretely sampled events. The amount of solids available for washoff during a given event is computed in SWMM using a constant accumulation rate of dust and dirt per length of gutter which is specific to each land use and the number of days since the last removal of dust and dirt by street cleaning or by a previous storm event. The SWMM input parameters used to produce the final calibrations for these processes in each basin are given in Table 4-3 by land use. Also presented are the SWMM default values, Figure 4-5 is an example of a water quality simulation output pollutograph.

Based upon the data obtained during the discrete sampling program and the results of the subsequent modeling effort, it was possible to develop several conclusions regarding the processes affecting stormwater quality in Tampa:

TABLE 4-3. STORMWATER QUALITY PARAMETERS USED IN CALIBRATION OF SWMM

Basin	Land use(1)	Percent land use in basin	Dust and dirt per 100 ft. gutter, lb	Soluble BOD per lb dust and dirt mg/gm	Non-soluble BOD(2)	Coliform multiplier(3)
J.L. Young	2	100	0.05	20	0.2	5.0
Wilder Ditch	1	51	0.04	50	0.6	1.0
	2	24	0.18	77		
	3	7	0.26	30		
	4	17	0.08	50		
North Jesuit	1	58	0.5	25	0.25	1.0
	4	42	2.2	38		
Charter & Harding	1	90	0.04	10	0.10	5.0
	4	10	0.08	10		
Norma Park	1	7	0.01	5.0	0.05	5.0
	3	93	0.02	7.7		
SWMM Default Values	1	-	0.7	5.0	0.05	1.0
	2	-	2.3	3.6		
	3	-	3.3	7.7		
	4	-	1.5	5.0		

1. Single family residential = 1, multi family residential = 2, commercial/institutional = 3, open space = 4.
2. Fraction of dust and dirt that is non-soluble BOD.
3. Multiplier used for coliform calibration.

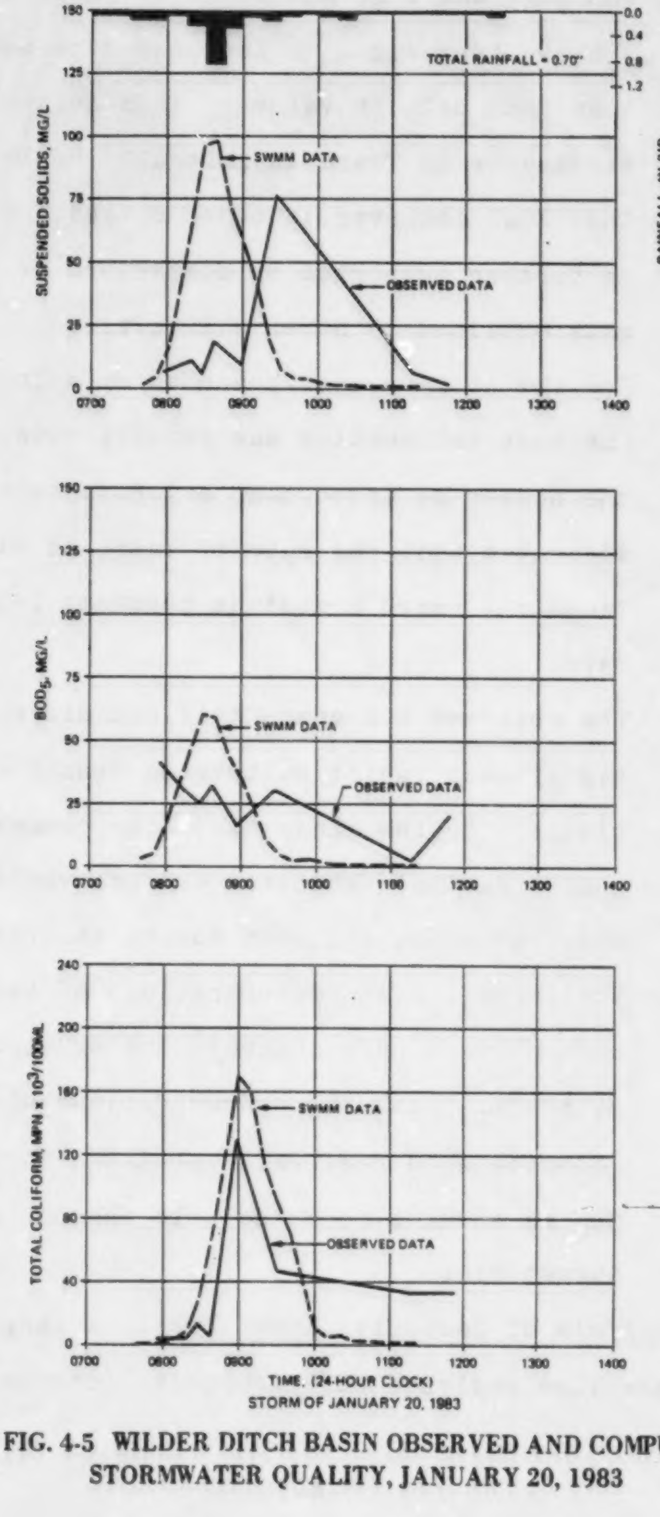

FIG. 4-5 WILDER DITCH BASIN OBSERVED AND COMPUTED STORMWATER QUALITY, JANUARY 20, 1983

1. The dust and dirt accumulation rates input to SWMM to achieve best fit with measured data were significantly less than default values. This suggests that stormwater in Tampa is generally of better quality than that observed in other cities. This conclusion is further supported by comparison of Tampa data with data obtained in other NURP cities
2. For the storm events preceded by a long dry period, the best calibration was usually obtained by reducing the number of antecedent dry days to six. This indicates that the rate of dust and dirt buildup in Tampa may reach a maximum constant level after several days.
3. The observed BOD concentrations did not exhibit significant variation between events or among the basins. On the other hand, the observed coliform concentrations exhibited extreme variability from event to event and even during an event.
4. The largest peak concentrations of suspended solids and coliform were observed during storms with intense rainfall. Peak BOD concentrations did not appear to coincide with peak rainfall intensity. The stormwater BOD is thought to be soluble and not associated with street dirt.

Analysis of Composite Event Data. A large number of storms were also analyzed on a composite (one sample per storm) basis to develop an understanding of the storm-to-storm variability of stormwater runoff and its quality. These analyses included calculating standard statistics for quality composite data from each basin, and linear regressions between storm characteristics and stormwater quality.

The standard statistics used to compare basin-to-basin quality variability were the mean value and standard deviation. Table 4-4 presents these statistics for TSS, BOD_5 and fecal coliform in each basin. The standard deviations indicate within-basin variability of approximately the same magnitude as the measurement means. Thus, the true mean values could be significantly different than the measured mean values.

Under these circumstances, only if the measured means from each basin differed by substantially more than the within-basin variability could valid relationships be derived establishing pollutant concentrations as a function of basin land use. However, the measured means are relatively close compared to the within-basin variability. This is shown in Table 4-5, which gives means and standard deviations for the pollutants, ascertained from the five mean values for each pollutant in Table 4-4. This suggests that stormwater quality may not be a direct function of land use in Tampa.

The independent and dependent variables analyzed through linear regression are shown in Table 4-6. The analysis was performed first for each basin. The results indicated that the

TABLE 4-4. OVERALL WATER QUALITY STATISTICS FOR THE MONITORING BASINS

Basin	Sample Size	Water Quality Parameter	Flow Weighted Mean Concentration	Standard Deviation of Measurements
1(1)	15	TSS(6)	42.3	36.9
1	15	BOD_5(6)	13.8	9.3
1	3	Fecal Coliform(7)	15700	4600
2(2)	18	TSS	33.5	22.7
2	18	BOD_5	16.6	15.5
2	2	Fecal Coliform	23300	7000
3(3)	18	TSS	70.9	94.4
3	18	BOD_5	16.7	7.7
3	2	Fecal Coliform	19200	24700
4(4)	15	TSS	29.1	34.2
4	15	BOD_5	10.5	9.0
4	3	Fecal Coliform	40600	18000
5(5)	15	TSS	19.4	15.1
5	14	BOD_5	11.7	6.3
5	3	Fecal Coliform	23000	11700

1. J.L. Young
2. Wilder Ditch
3. North Jesuit
4. Charter & Harding
5. Norma Park
6. mg/l
7. MPN/100ml

TABLE 4-5. STATISTICS OF THE OVERALL VALUES FOR THE FIVE MONITORING BASINS

Water Quality Parameter	Mean	Standard Deviation
TSS(1)	39.0	19.6
BOD(1)	13.9	2.8
Fecal Coliform(2)	24400	9600

1. mg/l
2. MPN/100 ml

coefficients were quite random and seemed to have little dependence on any particular factor. The data collected in the individual basins were subsequently pooled and analyzed as one data set. For the pooled data, in no case was there a correlation coefficient of 0.5 or greater (or -0.5 or less). The average coefficient for antecedent condition was about 0.2, while for all other independent variables it was about 0.1.

The results of the linear regression analysis suggest that stormwater quality in Tampa is not a predictable function of storm characteristics. Therefore, when area-wide flows and loads were developed, the total data set was utilized regardless of storm event characteristics.

Characteristics of Stormwater in Tampa. As a further step in characterizing stormwater runoff in Tampa, the stormwater quality data collected were compared with nationwide data (2). As shown in Table 4-7, for most of the parameters, Tampa runoff

TABLE 4-6. STORM CHARACTERISTICS AND POLLUTANTS USED IN LINEAR REGRESSIONS

Abbrv.	Storm Characteristics (independent variable)	Abbrv.	Pollutants (dependent variable)
ANT	Antecedent Condition (days)	BOD	Biochemical Oxygen Demand (mg/l)
DUR	Storm Duration (hours)	COD	Chemical Oxygen Demand (mg/l)
MIN	Maximum 15 Minute Intensity (in/hr)	TSS	Total Suspended Solids (mg/l)
AIN	Avg. Storm Intensity (in/hr)	OP	Ortho-Phosphate (mg/l)
TRV	Total Rainfall Volume (Mgal)	TP	Total Phosphate (mg/l)
QAV	Mean Runoff Flow Rate (mgd)	NIA	Nitrate (mg/l)
QPK	Peak Runoff Flow Rate (mgd)	NII	Nitrite (mg/l)
		TKN	Total Kjeldahl Nitrogen (mg/l)
		NH3	Ammonia (mg/l)
		RNH	Organic Nitrogen (mg/l)
		TN	Total Nitrogen (mg/l)
		CD	Cadmium (ug/l)
		CU	Copper (ug/l)
		PB	Lead (ug/l)
		ZN	Zinc (ug/l)

has lower concentrations, supporting the findings of the stormwater modeling discussed earlier in this section.

TABLE 4-7. COMPARISON OF TAMPA AND NATIONWIDE STORMWATER QUALITY DATA

Parameter	(Unit)	Mean concentration of pooled runoff data - Tampa	Mean concentration of pooled nationwide runoff data[1]	(No. of events)
BOD_5	(mg/l)	14	8	(375)
COD	(mg/l)	51	71	(709)
TSS	(mg/l)	40	108	(790)
Ortho-P	(mg/l)	0.14		
Total-P	(mg/l)	1.38	0.29	(737)
Nitrate	(mg/l)	1.89		
Nitrite	(mg/l)	0.02		
TKN	(mg/l)	1.24	1.5	(1,172)
Ammonia-N	(mg/l)	0.38		
Organic-N	(mg/l)	0.91		
Total-N	(mg/l)	1.66		
Cadmium	(ug/l)	2.49	18.2	(3,564)
Copper	(ug/l)	49	33	(790)
Lead	(ug/l)	57	142	(1,249)
Zinc	(ug/l)	59	134	(962)
Arsenic	(ug/l)	9		
Barium	(ug/l)	35		
Chromium	(ug/l)	8.4	33.4	(3,130)
Iron	(ug/l)	545		
Manganese	(ug/l)	14		
Mercury	(ug/l)	6.9		
Nickel	(ug/l)	13	24.4	(804)

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1. NURP Nationwide Report, First Draft, September 28, 1982 (2).

To determine where stormwater pollutant constituents come from, a comparison of rainfall (3) and stormwater quality was made, as shown in Table 4-8. This table suggests that stormwater copper and zinc comes primarily from rainfall. Stormwater TSS, total-P, lead and iron clearly derive significantly from surface buildup. For the other stormwater constituents, both sources may contribute substantially.

TABLE 4-8. SUMMARY OF MEAN RAINFALL QUALITY CONCENTRATIONS

Parameter	(Unit)	Mean concentration in pooled runoff data	Range of mean concentrations of rainfall quality (1)
BOD_5	(mg/l)	14	2 - 4
COD	(mg/l)	51	13 - 20
TSS	(mg/l)	40	4 - 11
Ortho-P	(mg/l)	0.14	0.04 - 0.19
Total-P	(mg/l)	1.38	0.10 - 0.23
Nitrate	(mg/l)	1.89	0.19 - 0.87
Nitrite	(mg/l)	0.02	0.004 - 0.014
TKN	(mg/l)	1.24	0.34 - 0.55
Ammonia-N	(mg/l)	0.38	<0.10 - 0.12
Organic-N	(mg/l)	0.91	0.34 - 0.53
Total N	(mg/l)	1.66	0.51 - 1.23
Cadmium	(ug/l)	2.49	0.55 - 1.5
Copper	(ug/l)	49	32 - 245
Lead	(ug/l)	57	3.8 - 7.3
Zinc	(ug/l)	59	92 - 189
Arsenic	(ug/l)	9	5 - 6
Barium	(ug/l)	35	<10 - 11
Chromium	(ug/l)	8.4	2.1 - 6.8
Iron	(ug/l)	545	9 - 45
Manganese	(ug/l)	14	3 - 8
Mercury	(ug/l)	6.9	1 - 2.8
Nickel	(ug/l)	13	1.5 - 4

1. Task II.3 Precipitation Quantity and Quality (3).

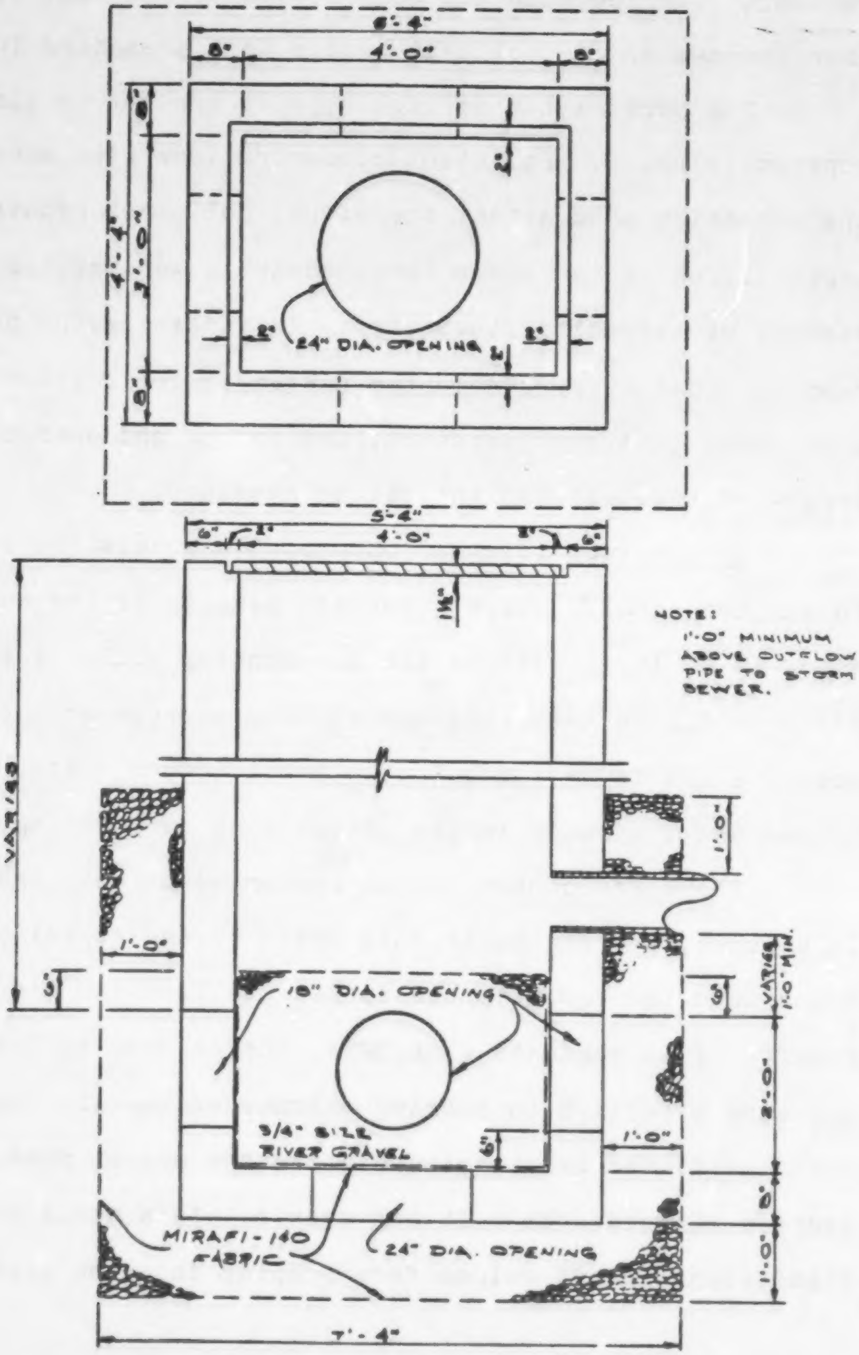
Analysis of Stormwater Controls

General. Four types of stormwater controls common in the City of Tampa were tested for runoff and pollutant removal capacity. Tests were performed on open bottom inlets, drain trenches and ditches to determine their capability to reduce hydraulic flow to storm sewers. These controls were tested during both dry and wet weather. A field program was conducted to measure stormwater hydraulics and pollutant loads associated with detention ponds in order to assess their capability for load removal. The location and description of the test sites for the four stormwater control test series were presented in Chapter 3.

The performance of each type of control is discussed separately below. Pollutant concentrations were measured only at the detention pond sites; therefore, pollutant removal capabilities of the other three controls were assessed based on removal of hydraulic flows only. In assessing the hydraulic removal capacity of stormwater controls, the following factors were taken into consideration: the inflow and outflow rates, the length of the test and the volume retained.

Open Bottom Inlets. Open bottom inlets (OBI) are designed to collect runoff, store a certain portion of the runoff and allow it to infiltrate to the surrounding soil. A typical OBI is shown in Figure 4-6. The approximate storage volume within the concrete box is 15 cubic feet. About 55 cubic feet of void volume are available in the gravel pack surrounding the box.

The three control tests performed on OBIs indicated that, in general, the available void space in the gravel pack within and around the concrete box is not utilized for storage of runoff. This suggests that OBIs, though easy to construct, are not very effective in storing stormwater runoff. It is possible that the filter fabric placed above the gravel pack (Figure 4-6) becomes clogged with silt and debris. This would prevent significant runoff volume from seeping into the gravel and may be


5'-4"
6"
4'-0"
6"
4'-4"
3'-0"
6"
6"
2"
2"
2"
2"
24" DIA. OPENING
5'-4"
6"
2"
4'-0"
2"
6"
1½"
NOTE:
1'-0" MINIMUM
ABOVE OUTFLOW
PIPE TO STORM
SEWER.
VARIES
1'-0"
1'-0"
3"
18" DIA. OPENING
3"
VARIES
1'-0" MIN.
2'-0"
3/4" SIZE
RIVER GRAVEL
3"
1'-0"
6"
1'-0"
MIRAFI-140
FABRIC
24" DIA. OPENING
7'-4"

SOURCE: CITY OF TAMPA

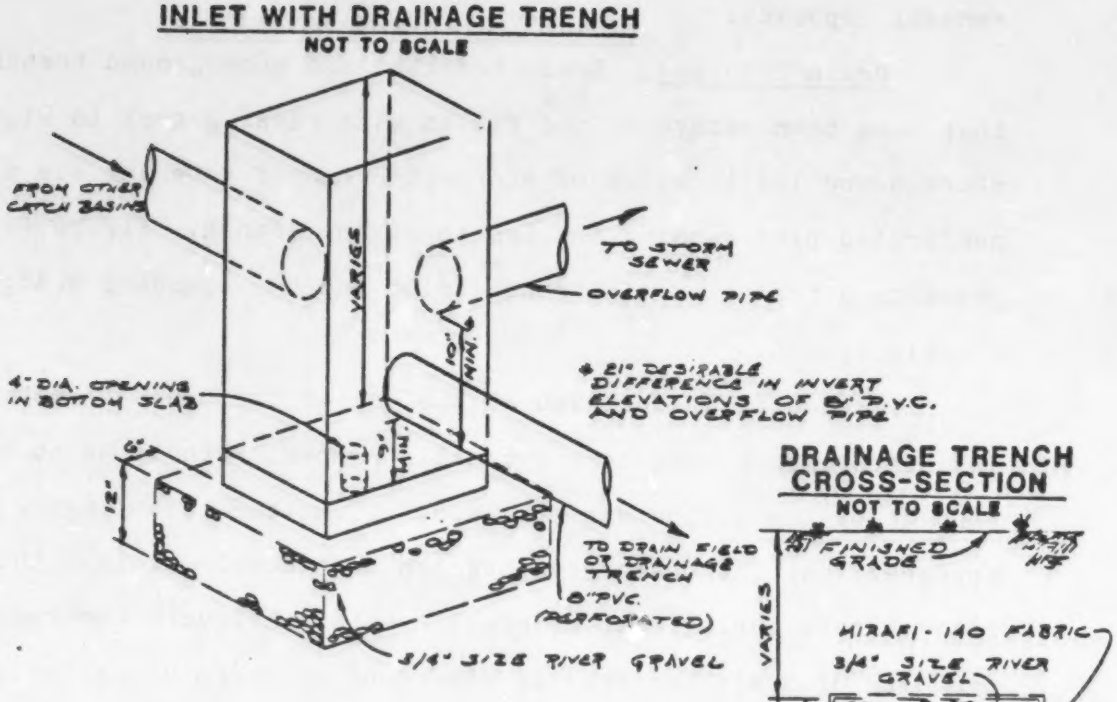
FIG. 4-6 TYPICAL DESIGN OF OPEN BOTTOM INLETS

the major reason that the OBIs demonstrated limited runoff removal capacity.

Drain Trenches. Drain trenches are underground trenches that have been excavated and filled with river gravel to allow storage and infiltration of stormwater runoff entering via a perforated pipe running the length of the trench. Figure 4-7 presents a typical drain trench inlet and the standard design of a drain trench.

The outflows observed at the end of the drain trenches indicated that a long-term hydraulic removal rate of 30 to 35 percent of the inflow was attained. Given the test lengths, it appeared that the drain trenches can continue to achieve this removal rate during most rainfall events. However, the removal capacity of drain trenches is dependent on storm characteristics. A drain trench may perform better during a low-intensity storm in wet weather than a high-intensity storm in dry weather. Potential for obstruction of the inlet pipe, hydraulic overloading from intense rainfall events, and saturated soil conditions further reduce the reliability of these controls.

The drain trenches performed consistently better than OBIs. The storage volume along drain trenches was utilized in most tests whereas storage around the OBIs was not. The primary reason for this may be the different design of the inlet chamber for these stormwater controls. The perforated pipe leading to the drain trench is located about 9 inches above the chamber


INLET WITH DRAINAGE TRENCH
NOT TO SCALE
FROM OTHER CATCH BASINS
VARIES
TO STORM SEWER
OVERFLOW PIPE
10" MIN.
± 21" DESIRABLE DIFFERENCE IN INVERT ELEVATIONS OF 8" P.V.C. AND OVERFLOW PIPE
4" DIA. OPENING IN BOTTOM SLAB
6"
9" MIN.
12"
TO DRAIN FIELD OF DRAINAGE TRENCH
8" P.V.C. (PERFORATED)
3/4" SIZE RIVER GRAVEL
DRAINAGE TRENCH CROSS-SECTION
NOT TO SCALE
FINISHED GRADE
VARIES
MIRAFI - 140 FABRIC
3/4" SIZE RIVER GRAVEL
12"
3'
1'-6"
3'
PERFORATED PIPE (8" P.V.C. SCH. 40)
LONGITUDINAL SECTION
NOT TO SCALE
INLET
8" P.V.C. CLEANOUT
2"±
MIRAFI - 140 FABRIC
1'
8" P.V.C. PERFORATED PIPE
3/4" RIVER GRAVEL
90° LONG SWEEP
VARIES
2' MIN.

SOURCE: CITY OF TAMPA

FIG. 4-7 TYPICAL DESIGN OF DRAIN TRENCHES

bottom. If debris did collect in the inlet box, the elevation of the drain trench pipe would allow it to continue functioning and transporting water down the drain trench.

OBI operation is dependent upon a clear surface above the gravel pack within the concrete box. If this clogs, little water can get through to the storage area and, subsequently, the soil around it. The data collected on both OBIs and drain trenches demonstrate the importance of routine maintenance and cleaning to insure correct functioning of these controls.

Ditch Systems. Ditch systems or swales are surface stormwater runoff controls that act as conveyors of runoff to the storm sewer and provide for infiltration of runoff to reduce flows and loads to the storm sewer. The inverts of regular ditches are normally level with the inverts of culverts that connect the ditches. Undercut ditches have an invert that is up to 0.5 feet below the adjacent culvert invert. This increases the storage area of the ditch. The test results at the ditch sites suggest that ditch system removal capacity for dry and wet weather conditions is equal to or superior to that of drain trenches. The long-term removal rate for regular ditches ranged from 27 to 69 percent of the inflow with an average of 43 percent. For undercut ditches the removal capacity was between 47 and 69 percent of the inflow with an average of 56 percent, an improvement over regular ditches. The ditches appear to be capable of sustaining these removal rates through most storm events.

Detention Ponds. A coupled system of detention ponds was selected for measurement and analysis of their capability to remove flow and pollutant loadings from stormwater runoff. The ponds, known as North Pond and South Pond, are located in Horizon Park.

The analysis of available data from the field measurement program for detention ponds indicated that they are capable of reducing peak runoff volume, particularly if dead storage volume is available. However such storage volume is not frequently available.

Available storage volume, storm intensity and duration all affect the pollutant removal capability of the detention ponds. It was found that the best pollutant removals occurred for two high intensity, short duration storms for which some dead storage was available in the ponds.

Pollutant removal was highest for those pollutants associated with particulate matter and lowest for pollutants that are present in soluble form. In-pond water samples usually reflected a "steady-state" concentration of most pollutant parameters over a 6-day sampling period.

Cost Analysis of Stormwater Controls. The available capital and O&M costs for the various stormwater controls are presented in Table 4-9. Cost estimates were based primarily on the City's own budget calculations and production estimates. These costs were compared to the removal benefits of each control.

TABLE 4-9. SUMMARY OF STORMWATER CONTROL COSTS(1)

Stormwater control	Capital unit costs ($)	Maintenance costs per year ($)
Open bottom inlets	1,175/OBI	34/OBI
Drain trenches	21.13/lin. ft.(2)	26.70/drain trench
Ditch systems/swales	5.69/lin. ft.	.06 - .10/lin. ft.
Retention/detention ponds	4.50/cu.yd. (excavation) 4.60/lin. ft. (fencing) 2.70/sq. yd. (sodding)	136/acre (mowing)

1. City of Tampa Fiscal Year 1983 estimates unless otherwise noted.
2. 1982 unit cost.

Table 4-10 presents a summary of long-term removal rates for the four stormwater controls. A cost/benfit analysis can be made for three of the four types of stormwater controls.

Open bottom inlets, though relatively inexpensive to build, are also relatively ineffective in reducing hydraulic load. Without an effective maintenance program, installing these controls will produce little benefit.

Costs and benefits of drain trenches and ditches are compared in Table 4-11 for a 150 ft. length of each of these controls. Using the costs of Table 4-9, drain trenches would be about four times more costly to build and maintain.

Capital costs of constructing the 2.9 acre South Pond are estimated at $86,000. Annual maintenance would be minimal in

TABLE 4-10. SUMMARY OF LONG-TERM REMOVAL RATES FOR VARIOUS STORMWATER CONTROLS

Stormwater control	Long-term hydraulic removal rate (% of inflow rate)(1)	Long-term pollutant removal rate (% of inflow load)
Open bottom inlets	0 - 5	0 - 5
Drain trenches	30 - 35	30 - 35
Ditch systems - regular	30 - 70	30 - 70
Ditch systems - undercut	50 - 70	50 - 70
Retention/Detention ponds	20 - 95	0 - 30 (soluble pollutants) 0 - 90 (particulate matter)

1. The hydraulic removal rate for detention ponds is the percent reduction or attenuation of the peak inflow rate.

TABLE 4-11. COMPARISON OF COSTS AND BENEFITS FOR A 150-FT. DRAIN TRENCH OR DITCH

Stormwater control	Capital costs ($)	O&M costs ($/year)	Expected load removal (%)
Drain trench	3,170	53.40	30 - 35
Ditch	850	15.00	30 - 70

comparison. The benefits associated with this pond include a potential of high removal of particulate matter present in the stormwater runoff. This removal is highly variable, however, and dependent on rainfall characteristics, antecedent condition, groundwater level, and in-pond available storage.

The relative area served by detention ponds and drain trenches or ditches was compared. South Pond receives inflow from about 75 acres which yields approximately 1 acre of detention area to 40 acres of drainage area. Assuming that the drain trench and ditch drainage areas are both 2 acres, about 40 drain trenches or ditches would be necessary to handle the load of one, two-acre pond. Construction of 40 drain trenches would cost approximately $127,000 with a yearly maintenance cost of $2,000, while 40 ditches would cost about $34,000 to construct. If land costs, surveying costs and other unaccounted for detention pond costs were less than $40,000 for a 2-acre pond, the pond would be economically preferable to 40 drain trenches. The cost of 40 ditches, however, was less than half of the accountable detention pond costs.

Projection of Study Area-Wide Loads

Development of a Stormwater Management Plan for the City required that stormwater flows and loads for the entire study area be quantified. This included:

1. Quantification of long-term stormwater runoff flows under design conditions, enabling assessment of needed flooding controls.
2. Quantification of long-term stormwater runoff pollutant loading under design conditions, enabling stormwater management planning for protection of receiving water quality. This fulfills two objectives:

a. The Stormwater Management Plan will be consistent with intended uses and desired quality of the receiving water.

b. The City of Tampa will have a basis with which to apply for State stormwater permits.

The Simplified Stormwater Management Model (S/SWMM)(4), was used to generate runoff volumes and pollutant loads for the wet (1969), average (1971) and dry (1974) years described earlier in this Chapter. Runoff and pollutant factors measured in a field study were used in the S/SWMM.

S/SWMM Inputs. S/SWMM calculates the quantity and quality of runoff, the effect that in-system storage may have on the amount of runoff, and the volume and pollution load of floodwaters. It requires specification of the rainfall record for the period of study and discretization of the overall drainage area into subareas. Each subarea must be assigned input values for the following:

1. Runoff coefficient (Rc),
2. Storage volume,
3. Outflow capacity,
4. Pollutant concentration.

The overall study area was represented in S/SWMM as 17 distinct subareas, each outflowing and overflowing directly to the River. Pertinent subarea characteristics are presented in Table 4-12. The subareas as input to the S/SWMM are shown in Figure 4-8.

TABLE 4-12. S/SWMM SUBAREA CHARACTERISTICS FOR THE EXISTING STORMWATER SYSTEM WITH PROJECTED LAND USE

Subarea No.	Subarea Name	Abbreviation	Runoff Area (acres)	Runoff Coefficient	Storage (mgal)	Outflow (mgal/hr)
1.	North Tampa	NT	929	.45	13.22	1.00
2.	Seaboard Coastline	SC	811	.56	8.32	2.00
3.	Sulphur Springs	SS	502	.47	.77	10.70
4.	River Bend	RB	873	.43	.62	27.43
5.	Forest Hills	FH	1524	.51	14.00	5.87
6.	Seminole Heights	SH	575	.43	.46	7.55
7.	Northwest Tampa	NW	1160	.39	31.44	.99
8.	Kirby Creek	KC	1131	.41	6.17	14.12
9.	Oak Grove	OG	469	.38	.95	20.25
10.	Hillsborough Ave.	HA	992	.46	3.54	5.97
11.	Wellswood	WW	666	.48	.55	18.19
12.	Sunshine Park	SP	856	.42	.48	17.96
13.	St. Joseph's	SJ	1022	.47	13.08	13.94
14.	Tampa Heights	TH	1071	.44	5.30	4.60
15.	West Tampa	WT	898	.43	.82	9.24
16.	University of Tampa	UT	657	.59	.58	33.00
17.	Downtown	DT	525	.63	.57	39.59

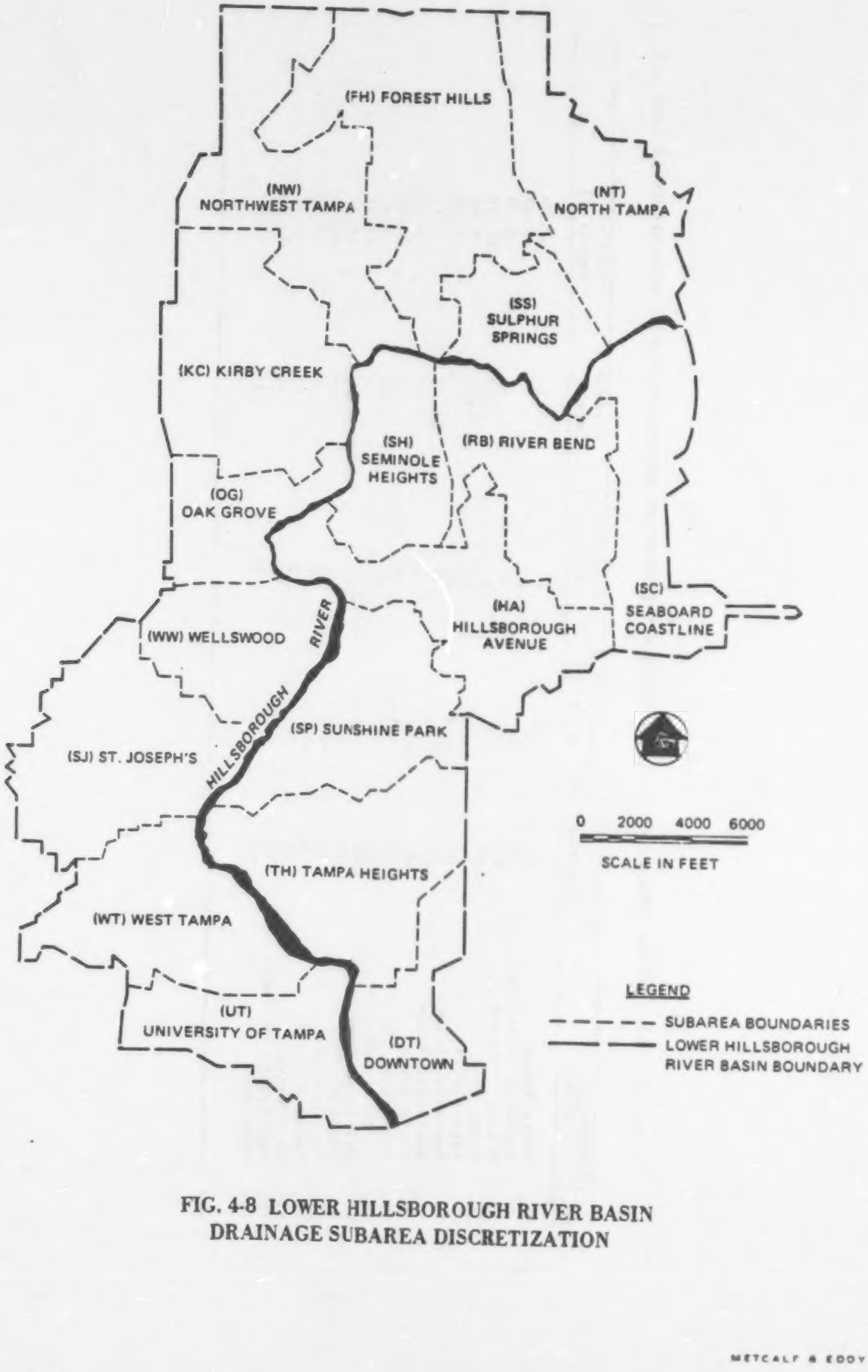

(FH) FOREST HILLS
(NW) NORTHWEST TAMPA
(NT) NORTH TAMPA
(SS) SULPHUR SPRINGS
(KC) KIRBY CREEK
(SH) SEMINOLE HEIGHTS
(RB) RIVER BEND
(OG) OAK GROVE
(SC) SEABOARD COASTLINE
(HA) HILLSBOROUGH AVENUE
(WW) WELLSWOOD
HILLSBOROUGH RIVER
(SP) SUNSHINE PARK
(SJ) ST. JOSEPH'S
(TH) TAMPA HEIGHTS
(WT) WEST TAMPA
(UT) UNIVERSITY OF TAMPA
(DT) DOWNTOWN
0 2000 4000 6000
SCALE IN FEET
LEGEND
SUBAREA BOUNDARIES
LOWER HILLSBOROUGH RIVER BASIN BOUNDARY

FIG. 4-8 LOWER HILLSBOROUGH RIVER BASIN DRAINAGE SUBAREA DISCRETIZATION

The runoff coefficients of each subarea were calculated based on land usage and the runoff coefficients derived for various land uses as given in Table 4-13.

Pollutant concentrations that were representative of average stormwater from the entire lower Hillsborough River basin were used in the model. A determination of the probability distributions of the composite data for each pollutant was made, and the mean and standard deviation of these distributions were calculated. It was found that BOD_5 and fecal coliforms were normally distributed and that TSS was lognormally distributed. The median value for these pollutants was used in S/SWMM simulation. Due to the high variability of the quality data, S/SWMM was also run with quality concentrations corresponding to a confidence interval of one standard deviation around the median values. These values are listed in Table 4-14.

S/SWMM Check. Before S/SWMM was used to generate the flows and loads for the study area, the simulations were checked against the field measurement data. Every composite event with both TSS and BOD_5 data was simulated. The observed and predicted outflows correspond within a range of 61 to 122 percent. For each basin and each pollutant, the observed total loads fell within the predicted total load range.

It should be noted that the simulation of stormwater flows and loads on an area-wide basis relies on the assumption that the measurements conducted at specific locations can be used to

TABLE 4-13. RUNOFF COEFFICIENTS DERIVED FOR VARIOUS LAND USES

Land use	Runoff coefficient
Commercial/Industrial	0.90
High Density Residential	0.68
Medium Density Residential	0.53
Low Density Residential	0.37
Institutional	0.31
Open/Natural	0.20

TABLE 4-14. RANGE OF QUALITY CONCENTRATIONS TO BE SIMULATED IN S/SWMM

	Minus one standard deviation	Median	Plus one standard deviation
TSS (mg/l)	7.9	22.2	62.4
BOD_5 (mg/l)	3.8	14.1	24.4
Fecal Coliforms (MPN/100 ml)	9,030	25,400	41,800

characterize the entire study area. This has been demonstrated for hydraulic as well as water quality data. The following facts support the assumption of transferability of data within the study area:

1. Flow data at monitored basins have been shown to be related to impervious area.
2. Variation among basins in stormwater quality has been shown to be statistically uncorrelated to storm or

basin parameters. Therefore this variability is expected to be representative of variability for the entire study area, regardless of storm and basin conditions.

3. The S/SWMM with measured runoff coefficients and predicted quality was checked against measurements and was found to perform well.

Analysis of S/SWMM Predictions. Table 4-15 characterizes flooding in the lower Hillsborough River basin for the three years of simulation. Flooding volume increased with increasing hydraulic load. The rate of this increase, however, diminished as annual load increases. Note that average year flood volume is 3.0 times that of the dry year, while wet year flood volume is only 1.2 times that of the average year.

A review of the model results indicated that storms with hourly intensities greater than 1 inch/hour were the major cause of flooding. For the wet year, there were seven storms with hourly intensities greater than 1 inch per hour. These were responsible for about 90 percent of the total flooding volume for most of the subareas. For the average year, there were five hourly intensities greater than 1 inch per hour. Two of these intensities occurred on consecutive hours during the event of September 9, 1971. This storm resulted in the worst flooding in the entire three years of record for most subareas. Flooding during the dry year was relatively limited; only one event had an

TABLE 4-15. HYDROLOGIC SUMMARY FOR LOWER HILLBOROUGH RIVER BASIN IN WET, AVERAGE AND DRY YEARS

	Wet	Average	Dry
Flooding volume (MG)	364	319	102
Runoff volume (MG)	10,100	8,660	6,340
$\frac{\text{Flood Volume}}{\text{Runoff Volume}} \times 100$ (%)	3.6	3.7	1.6
Sum of subarea flood events	56	42	23
Total number of events	110	95	78
$\frac{\text{Flooding events}}{\text{Total number of events}} \times 100$ (%)	13	11	9
Total annual rainfall (inch)	59.2	46.3	33.9

hourly intensity greater than 1 inch per hour, and this caused most of the flooding.

A breakdown of flood volume by subarea and simulation year is given in Figure 4-9. The largest amount of flooding, in decreasing order, occurs in the Forest Hills, Seaboard Coastline, Tampa Heights and Hillsborough Avenue subareas. Drainage from Forest Hills is limited by capacity; however, a Florida Department of Transportation retention facility helps alleviate flooding. The Seaboard Coastline subarea is not residential, and is therefore generally not a problem area for the City. Tampa Heights and Hillsborough Avenue are subareas where current flooding problems exist and complaints are received by the City.

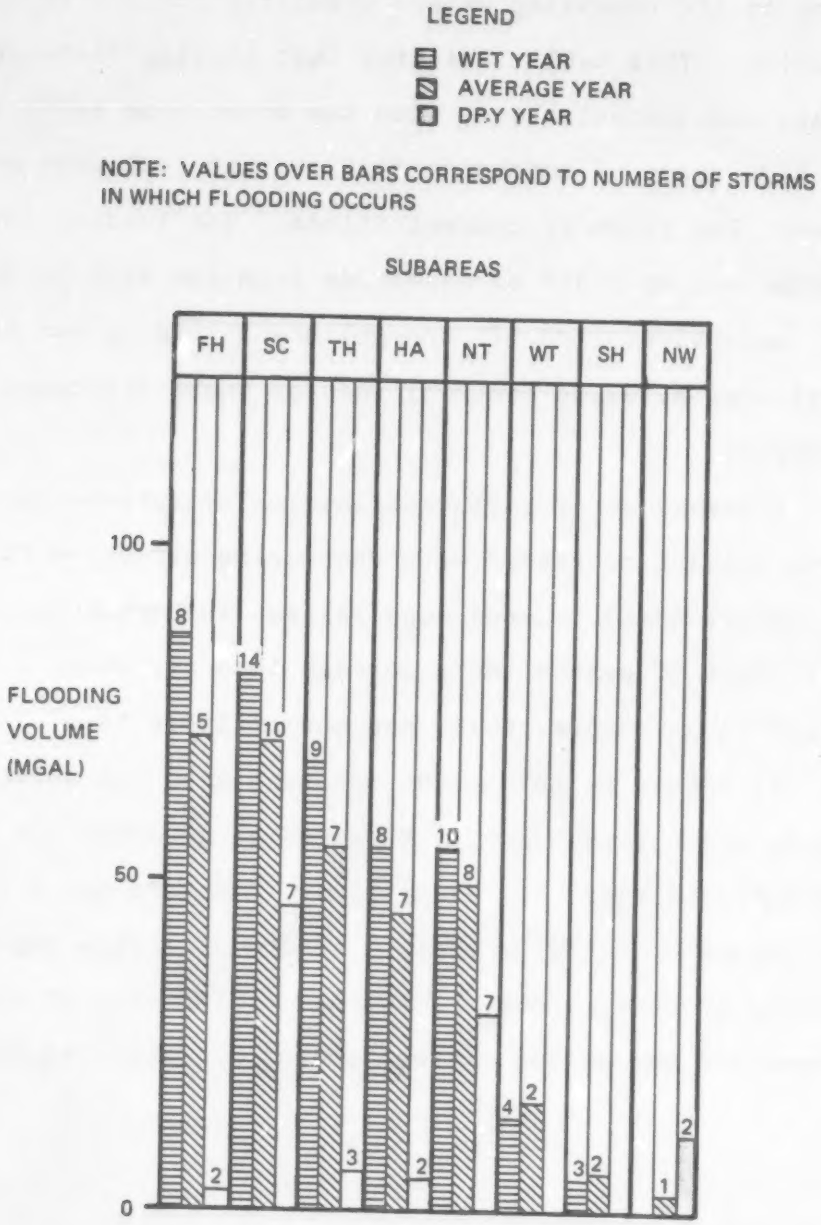


FIG. 4-9 FLOODING SUMMARY BY SUBAREA FOR WET, AVERAGE AND DRY YEARS

Table 4-16 summarizes the probable range of pollutant loading to the receiving waters predicted for the three years of simulation. This table indicates that loading increased with rainfall and concentration, with the worst case being the wet year, high range of concentrations and the best case being the dry year, low range of concentrations. The loading increased approximately an order of magnitude from the best to the worst case. Reduction of runoff and pollutant loading can be accomplished by using controls such as those discussed earlier in the Chapter.

A breakdown of pollutant loading by subarea indicated that subarea loading decreased with decreasing effective runoff area (Rc x Runoff Area). Loads were largest in Forest Hills, which has at least 30 percent more acreage than any other subarea, and smallest in Oak Grove, which has the smallest Rc.

It should be noted that subareas with the worst flooding problems are not necessarily those which generate the highest pollutant loadings. For example, St. Joseph's had a relatively minor amount of flooding because of its available capacity and in-system storage. However, because of the size of this subarea, it generated the second largest amount of pollution load.

TABLE 4-16. POLLUTANT RUNOFF TO LOWER HILLSBOROUGH RIVER IN WET, AVERAGE AND DRY YEARS

	BOD (lbs x 10^4)			TSS (lbs x 10^4)			Fecal coliform (organisms x 10^{14})		
	H(1)	M	L	H	M	L	H	M	L
Wet	206	119	32	528	188	67	160	97	35
Average	176	102	27	451	160	57	137	83	30
Dry	129	75	20	330	117	42	100	61	22

1. H = High range, one standard deviation above median, M= median, L = low range, one standard deviation below median.

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1. Metcalf & Eddy, Inc., University of Florida, and Water Resources Engineers, Inc., Storm Water Management Model, Volumes I-IV, Environmental Protection Agency, Water Quality Office, 1971.

2. Environmental Protection Agency, First Draft, Final Report of the Nationwide Urban Runoff Program, Water Planning Division, September, 1982.

3. Priede-Sedgwick, Inc., Precipitation Quantity and Quality Data Task II.3, Tampa Nationwide Urban Runoff Program Study Phase II, February, 1983.

4. Metcalf & Eddy, Inc., Development and Application of a Simplified Stormwater Management Model, Environmental Protection Agency, Water Quality Office, 1976.

CHAPTER 5

LOWER HILLSBOROUGH RIVER SAMPLING

General

In conjunction with their participation in the Nationwide Urban Runoff Program (NURP), the City of Tampa sponsored a study of the lower Hillsborough River estuary (1). The intent of the study was to better characterize water quality in the estuary and to collect and analyze additional data to determine the impact of stormwater runoff on this water body.

In Phase I of the Tampa NURP study an assessment was made of the most recent water quality data to identify possible problems and to investigate a cause and effect relation between urban stormwater runoff and changes in water quality in the receiving waters near Tampa (2). Phase I of the NURP Study focused on analysis of data collected in Hillsborough Bay, the lower Hillsborough River, and Hillsborough Reservoir. The three receiving water bodies were analyzed to determine which was best suited for the study of urban stormwater runoff impacts. To do this, each water body was tested against several criteria. The results indicated that the effects of stormwater runoff would be best studied in the lower Hillsborough River, followed by the Reservoir, and the Bay.

The receiving water data available for the Phase I study consisted primarily of single monthly samples. These data were not adequate to establish a cause and effect relationship between urban runoff and receiving water problems. A more detailed data collection and analysis program was therefore developed for the lower Hillsborough River to be conducted simultaneously with the NURP Project. In this way the characterization of stormwater flows and loads from urban drainage basins performed during the NURP study would be coordinated with measurements and analysis of the receiving waters.

In this Chapter the collection and analysis of data for the lower Hillsborough River study are described in order to characterize the influence of various parameters and their interaction on the lower Hillsborough River hydrodynamics and water quality.

Continuous Data

The lower Hillsborough River in Tampa is an estuary, and its water quality is dependent on many interactive phenomena. These include the following:

1. Freshwater discharge (and its quality) from the Hillsborough Reservoir.
2. Tidal variation and water quality in Hillsborough Bay.
3. Stormwater runoff from the City of Tampa.

The freshwater discharge from the Hillsborough Reservoir varies seasonally and is highest in August and September during the wet season. During this period estuarine characteristics disappear almost entirely from the upper segments of the River. During times of very low discharge from the Reservoir, saline intrusion occurs and the quality of Hillsborough Bay affects River quality.

In order to evaluate the effects of various hydrologic and climatological conditions on the lower Hillsborough River, continuous time-series data were collected during a 15-month period (July 1981 to September 1982) at Platt Street, Columbus Drive, Sligh Avenue and 22nd Street. These four stations are shown in Figure 5-1. The tidal stage was measured using continuous recording meters and digitized at fifteen minute intervals. In the same manner temperature, dissolved oxygen and conductivity were continuously measured and digitized at hourly intervals. At each station, meters were located at approximately mid-depth in the center of the stream.

The time series data were digitized and stored on computer tape by the USGS. Time series computer plots were generated and statistical analyses were performed on the data. A summary of the results of the analysis follows.

Tides. The tidal wave generally has an amplitude of 2 to 4 feet and progresses up the River without interference until upstream of Sligh Avenue. A discrepancy of up to 2 feet was noted at times between gage heights at 22nd Street and at the other stations, primarily during periods of high freshwater releases from Hillsborough Reservoir. The City of Tampa performed a physical inspection of the River between Sligh Avenue and 22nd Street which led to the finding of several shoal areas that could impede the decrease in depth on ebb tide. This knowledge was important to the hydrodynamic calibration of the River model which will be discussed in Chapter 6.

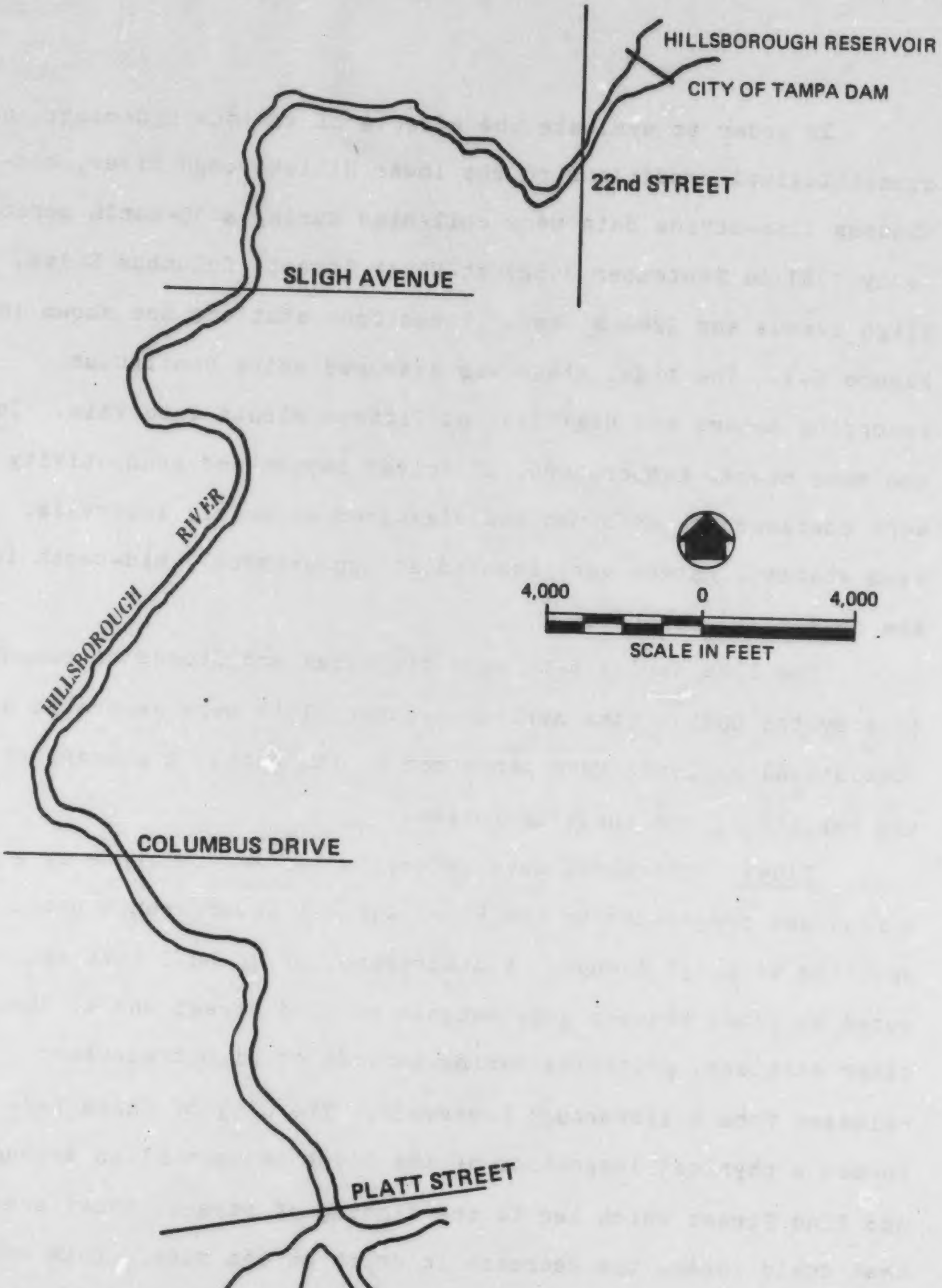

HILLSBOROUGH RESERVOIR
CITY OF TAMPA DAM
22nd STREET
SLIGH AVENUE
HILLSBOROUGH RIVER
4,000
0
4,000
SCALE IN FEET
COLUMBUS DRIVE
PLATT STREET
HILLSBOROUGH BAY

FIG. 5-1 PLAN OF LOWER HILLSBOROUGH RIVER SHOWING CONTINUOUS DATA SAMPLING LOCATIONS

Freshwater Flow. The two main sources of freshwater flow to the lower Hillsborough River are the controlled releases from the Hillsborough Reservoir and the natural flow from Sulphur Springs located approximately 2 miles downstream of the Dam.

Freshwater releases from Hillsborough Reservoir, a source of water supply for the Tampa area, are dependent on seasonal Reservoir levels and meteorological conditions. Table 5-1 presents the average monthly discharge from the Hillsborough Reservoir for the period covering the continuous data collection program. The monthly average discharge from Sulphur Springs is also shown in Table 5-1. During dry periods, water is pumped from Sulphur Springs to the Reservoir at a rate of 25 cfs to supplement the public water supply.

Temperature. River temperatures vary little on an hourly or daily basis during periods of moderate (0-500 cfs) freshwater releases from Hillsborough Reservoir and moderate temperatures. Figure 5-2 presents a comparison between temperature and gage height at Platt Street in July and August, 1982. Freshwater releases from the Reservoir ranged from 400 to 1,600 cfs during this period. It appears that the freshwater releases were several degrees cooler than water from Hillsborough Bay since the River temperature increases as the tide rises and decreases as it falls. Upstream stations exhibited similar variations but to a lesser extent. This is because the proportion of Bay water decreases heading upstream.

TABLE 5-1. MONTHLY AVERAGE FRESHWATER FLOW (CFS) TO THE LOWER HILLSBOROUGH RIVER(1)

Month	Hillsborough Reservoir	Sulphur Springs
July 1981	< 1.26	21.5
August	53.50	44.1
September	806.	49.6
October	44.40	43.7
November	0.65	41.4
December	0.79	17.6
January 1982	155.	44.1
February	134.	39.9
March	174.	36.9
April	163.	41.0
May	1.57	41.3
June	200.	48.6
July	839.	47.8
August	984.	48.1
September	1,124.	46.4

1. Source: USGS

One full year of temperature records (October 1, 1981 through September 30, 1982) was analyzed to obtain a frequency distribution of temperature at each of the four stations. The average temperature is about 25°C in most of the River.

Salinity. Figure 5-3 is a plot of tidal height, salinity and freshwater flow at Columbus Drive. This Figure illustrates

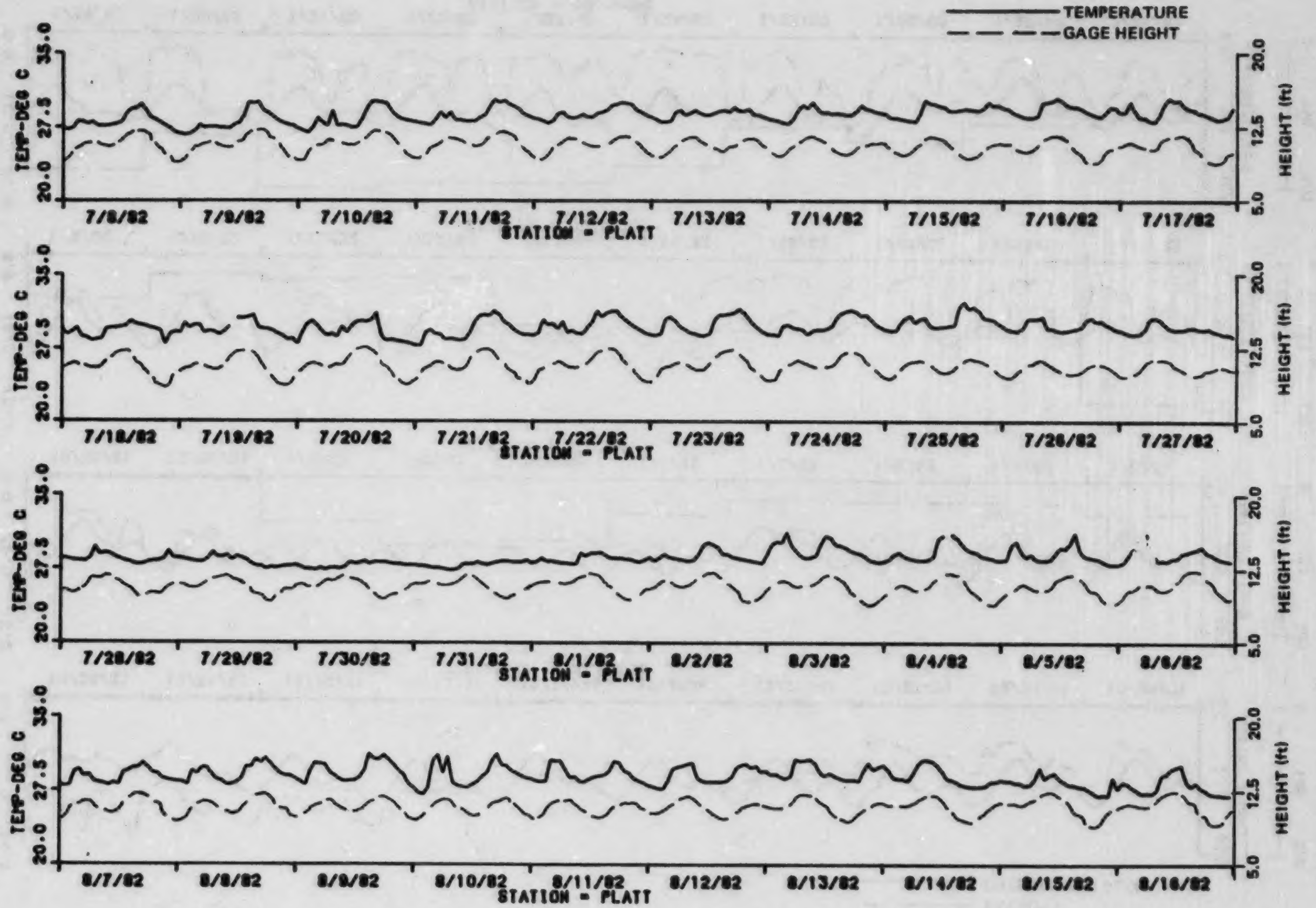


FIG. 5-2 TEMPERATURE AND GAGE HEIGHT AT PLATT STREET
JULY - AUGUST, 1982

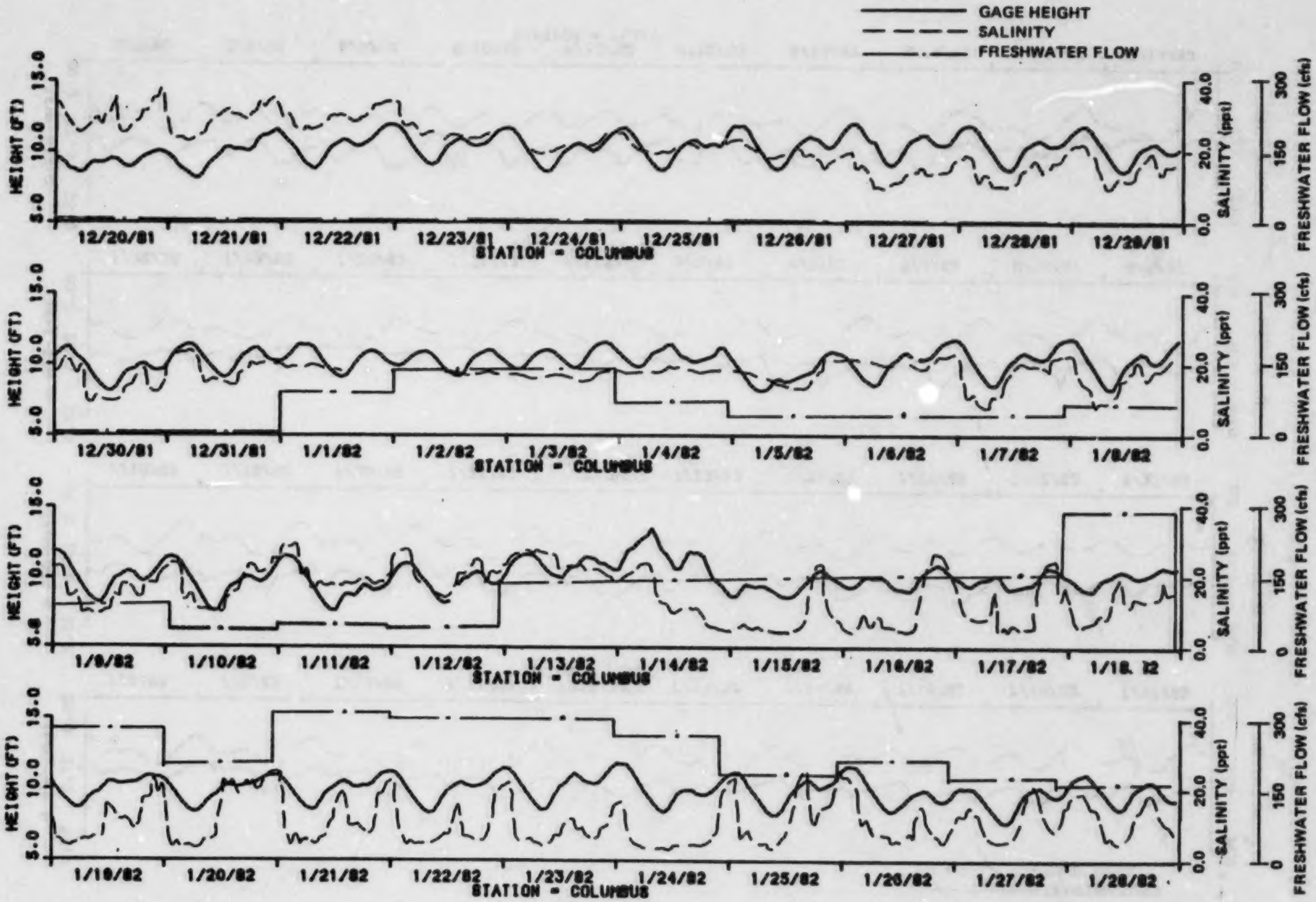


FIG. 5-3 GAGE HEIGHT, SALINITY, AND FRESHWATER FLOW AT COLUMBUS DRIVE
DECEMBER 1981 - JANUARY 1982

that the penetration of the saline wedge is related directly to the tidal motion in Hillsborough Bay and to the freshwater flow from the Reservoir. The short term (hourly) salinity variations are caused by the ebbing and flooding of the tide. River salinity rises during flood tide, as Bay water moves upriver, and falls during ebb tide.

Long term changes in salinity are caused by freshwater discharge. Through the end of December 1981, there was essentially no discharge from the Reservoir. In early January the freshwater flow increased to about 150 cfs, decreased for several days, then increased again. The effect on the salinity concentrations is clearly evident.

A frequency distribution of salinity at each station was obtained for one year of record from October 1, 1981 through September 30, 1982. The results of this analysis are presented in Table 5-2. This Table shows a wide range of salinity concentrations at all four stations including the ocean boundary (Platt Street). The very low salinity concentrations reflect conditions of high freshwater flow on the order of 1,500 cfs that occurred in early and late July, 1982. The average salinity concentrations are quite low at Sligh Avenue and 22nd Street, indicating that significant saline intrusion at mid-depth in these reaches of the River is rare.

Dissolved Oxygen. The tidal variation and DO concentrations at Columbus Drive are shown in Figure 5-4. On flood tide the DO levels generally drop, and on ebb tide they rise. It was

TABLE 5-2. SUMMARY STATISTICS OF SALINITY (PPT) FOR THE LOWER HILLSBOROUGH RIVER

Station	Range of Salinity (ppt)	Percentile exceedance salinity (ppt) 90%	50% (avg)	10%
Platt Street	.03 - 34.3	4.5	19.4	27.1
Columbus Drive	.09 - 38.0	2.0	13.3	24.1
Sligh Avenue	.09 - 22.5	0.3	1.9	16.4
22nd Street	.08 - 15.6	0.2	0.6	6.3

observed that the lowest DO concentrations occur in the bottom layer of the saline wedge. During ebb tide the wedge travels downstream and DO levels rise. The diurnal variations in DO levels indicate algal photosynthetic activity.

No freshwater was released from the Reservoir from May 29 through June 23, 1982. On June 24, 1982 the Reservoir began discharging at 188 cfs and increased to over 700 cfs on subsequent days. The dissolved oxygen concentrations during this period increase slowly and the diurnal and tidal variations are suppressed.

A frequency distribution of dissolved oxygen was obtained for each station for the period from October 1, 1981 through September 30, 1982. The exceedance frequency plot for dissolved oxygen at all stations is shown in Figure 5-5. This Figure shows

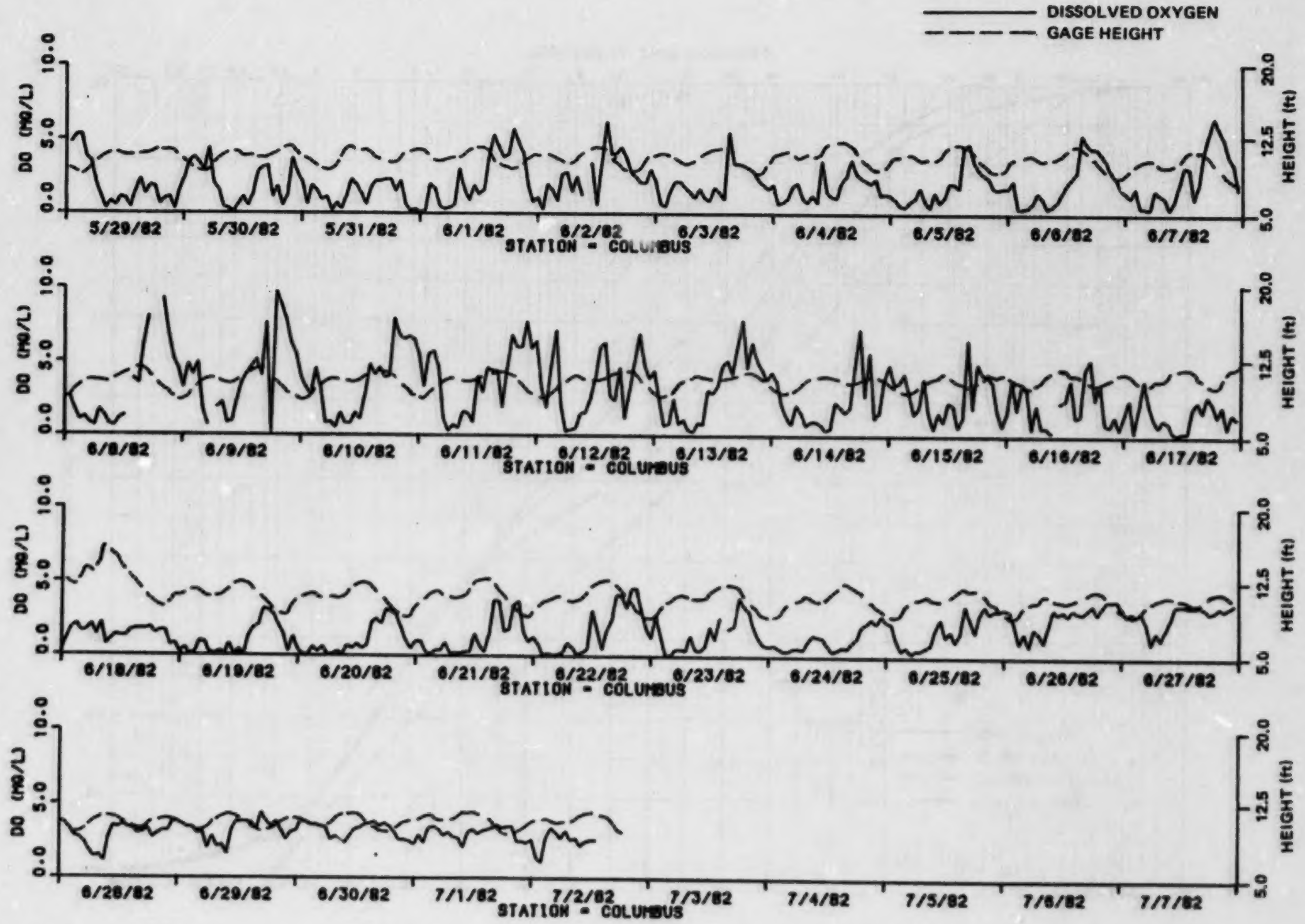


FIG. 5-4 DISSOLVED OXYGEN AND GAGE HEIGHT AT COLUMBUS DRIVE
MAY - JULY, 1982

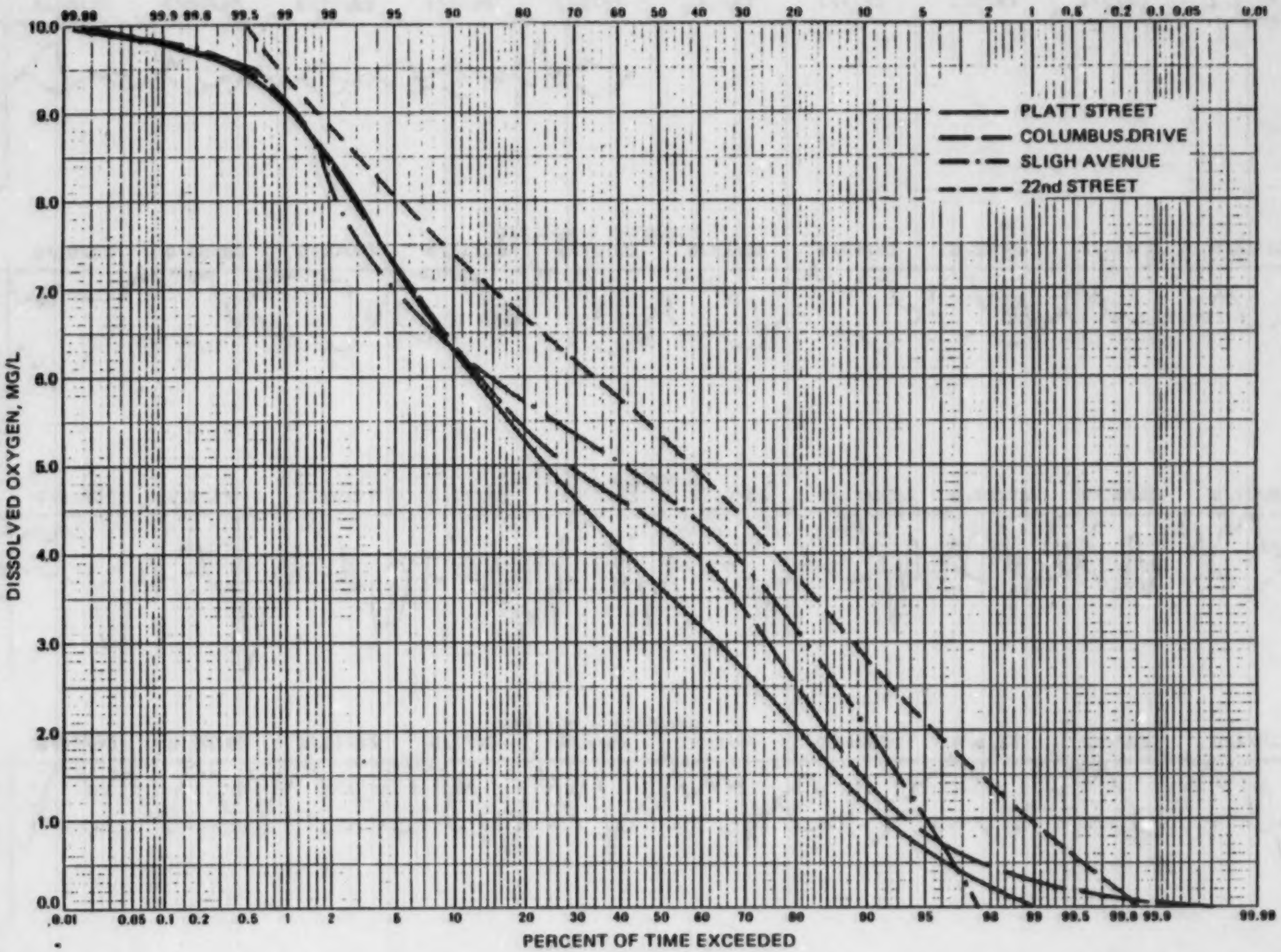


FIG. 5-5 EXCEEDANCE FREQUENCY PLOT OF DISSOLVED OXYGEN

that a mid-depth dissolved oxygen level of 4.0 mg/l is not met 60 percent of the time at Platt Street and that the 50 percentile or average DO concentration at mid-depth is approximately 3.6 mg/l. Statistics for the remaining stations are summarized in Table 5-3. These results indicate that the average mid-depth DO concentrations tend to decrease as one travels downstream on the lower Hillsborough River. A similar observation was made during the Phase I study of the NURP program (2).

A dissolved oxygen frequency analysis was also performed for periods of low, moderate and high freshwater discharge from Hillsborough Reservoir. The average daily Reservoir discharges during the 15-month data collection period were examined. Months of low (< 10 cfs) freshwater discharge from the Hillsborough Reservoir were separated from those with a discharge of 10 to 400 cfs and those with discharges greater than 400 cfs. The corresponding DO data were analyzed separately. The results of this analysis are presented in Table 5-4. During periods of low freshwater discharge the average DO levels are highest at Platt Street and lowest at Sligh Avenue where the saline penetration is prominent due to low freshwater flows. DO concentrations are usually lower in the vicinity of the saline wedge. Moderate freshwater releases significantly improve average DO concentrations at Columbus Drive, Sligh Avenue and 22nd Street whereas average DO concentrations at Platt Street remain the same. Prolonged freshwater discharges greater than 400 cfs result in very low DO concentrations at Platt Street. The high

TABLE 5-3. SUMMARY STATISTICS FOR DISSOLVED OXYGEN FREQUENCY DISTRIBUTION

Station	Range of DO concentrations (mg/l)	Average (50 percentile) DO concentration (mg/l)	Percent of time DO concentration is < 4.0 mg/L
Platt Street	0.0 - 10.0	3.6	59
Columbus Drive	0.0 10.0	4.3	41
Sligh Avenue	0.0 - 10.0	4.8	31
22nd Street	0.0 - 16.0	5.2	24

freshwater flows limit the saline penetration to this region. Columbus Drive, Sligh Avenue and 22nd Street exhibit a lower average DO level at high freshwater flows than during moderate flows. Reservoir water is usually released from the lower gates of the Dam during high freshwater flows. Low DO concentrations in the bottom waters of the Reservoir contribute to lower overall DO concentrations in the River during high flow periods.

As described in the Phase I Report (2), the estuarine circulation pattern in the lower River is such that fresh water flows downstream and the denser saline water moves upstream near the bottom of the water column. The net upstream movement of the bottom layer which brings in organic solids from the Bay, in combination with the large sediment oxygen demand in the lower

TABLE 5-4. DISSOLVED OXYGEN STATISTICS IN THE LOWER HILLSBOROUGH RIVER AS A FUNCTION OF FRESHWATER FLOW FROM HILLSBOROUGH RESERVOIR

Station	Low freshwater release (0-10 cfs) (1)		Moderate freshwater release (10-400 cfs) (2)		High freshwater release (>400 cfs) (3)	
	Average DO concentration (mg/1)	Percent of time DO<4.0mg/1	Average DO concentration (mg/1)	Percent of time DO<4.0 mg/1	Average DO concentration (mg/1)	Percent of time DO<4.0 mg/1
Platt Street	4.1	48	4.1	47	3.0	80
Columbus Drive	3.4	59	4.7	24	4.2	40
Sligh Avenue	2.8	79	5.2	16	4.7	13
22nd Street	3.8	56	6.7	6	5.2	5

1. October, November, December, 1981; May, June, 1982.
2. January through April, 1982.
3. July through September, 1982.

River and limited vertical mixing, most likely cause the observed DO sag to occur at the upstream end of the saline intrusion.

It should be noted that the continuous data were collected only at mid-depth. In a later section, the vertical variation of salinity and DO is analyzed. This leads to further understanding of the observed salinity and DO variations in the lower River.

Stormwater Runoff. The short and long term impacts of stormwater runoff on the water quality of the lower Hillsborough River are also of interest. The continuous data indicated that during periods of low freshwater flows, the DO levels tend to increase at Sligh Avenue and 22nd Street stations after a storm event and either do not change or decrease slightly at the Columbus and Platt stations. The salinity at all stations decreases after storms during low freshwater flow periods. The flushing provided by the stormwater runoff is significant enough to dilute the salinity and improve the DO in the stagnant upstream areas.

During periods of moderate to high freshwater flows, the four stations exhibit little or no change in DO or salinity concentrations after a rain storm. The Reservoir releases alone tend to reduce the salinity concentrations as discussed earlier. In no case was there a clearly identifiable DO sag caused by storm events.

Synoptic Surveys

Several synoptic surveys were conducted during this River study. Synoptic surveys are intensive field surveys designed to evaluate the short-term (inter-tidal) variations of water quality in the lower Hillsborough River. The data are collected at several stations (several depths per station), several times per day.

The purpose of these surveys was to compare water quality conditions under different hydrologic conditions. In particular, an evaluation of the effect of stormwater on the lower River was desired. The data are used to supplement the continuous data discussed earlier. Finally, these measurements were used to calibrate and verify the hydrodynamic and water quality river/estuary model as presented in Chapter 6.

Five synoptic surveys were conducted in the lower Hillsborough River. The surveys are summarized in Table 5-5. The surveys were designed to collect data during the following conditions:

1. A dry weather period with no freshwater flow.
2. A dry weather period with intermediate freshwater flow.
3. A dry weather period with large freshwater flow.
4. Before, during and after a storm event with no freshwater flow.
5. Before, during and after a storm event with high freshwater flow.

The data from these programs are used to evaluate the impact of stormwater on the lower River. In addition, the effect of freshwater flow from upstream can be demonstrated.

TABLE 5-5. LOWER HILLSBOROUGH RIVER SYNOPTIC SURVEYS

Date	Survey length (days)	Hillsborough Reservoir release (cfs)	Rainfall (inches)	Purpose	Conducted by
9/11/81	1/2	1000	-	dry weather high freshwater flow	USGS, City of Tampa
4/29 - 30/82	1	120	-	dry weather moderate freshwater flow	USGS, City of Tampa
11/4 - 7/82	3	300-500	1.5	wet weather high freshwater flow	Mote Marine Lab, Inc.
2/1 - 4/83	3	0	3.6	wet weather no freshwater flow	Mote Marine Lab, Inc.
5/16 - 17/83	1	0	-	dry weather no freshwater flow	Mote Marine Lab, Inc.

During each survey, data were collected at four stations. These were the same stations as used for the continuous data collection, as were shown in Figure 5-1. A summary of the types and frequencies of data collected during each program is given in Table 5-6.

Field crews occupied each station for the entire time of the survey. Tidal height measurements were obtained from continuous recording tidal gages at intervals of 15 minutes at each station. Vertical profiles of stream velocity, temperature, dissolved oxygen and conductivity were taken at several depths at

TABLE 5-6. DATA COLLECTED DURING SYNOPTIC SURVEYS

Data type	Method of collection	Interval of collection	Location of collection	Parameters
Tide	Continuous Recording Gage	15 minutes	4 stations	
Vertical Profile	Profiling meters	30 minutes-4 hours	4 stations, 3-5 depths	Velocity Temperature Conductivity DO
Water Quality	In-situ grabs	4-6 hours	4 stations surface and bottom	BOD_5 TSS Fecal Coliform Total Coliform
Water Quality	In-situ grabs	4-6 hours composited every 18 hours	4 stations composite of surface, mid-depth and bottom	BOD_5 COD TKN NO_2-N NO_3-N NH_3-N PO_4-P Total P TSS VSS Chlorophyll a Aluminum Copper Lead Zinc

mid-stream. Samples for BOD_5, total suspended solids and fecal and total coliform were taken every 4 to 6 hours at all four stations. Additional water samples were composited over 18-hour periods and analyzed for nutrients, chlorophyll a and metals. Water quality samples were also obtained at Sulphur Springs during some of the surveys.

As an example, the data collected during the November 4-7, 1982 survey is briefly described. This survey covered a 72-hour period with freshwater discharges from the Reservoir varying from 300 to 500 cfs. About 0.7 inches of rain fell immediately prior to the commencement of the survey. Eight hours later an additional 0.6 inch of rain was recorded.

Saline water was present only at Columbus Drive and Platt Street since the upstream stations were well-flushed by the freshwater discharge. DO concentrations were near saturation levels at Sligh Avenue and 22nd Street and varied from 2 to 6 mg/l at the Columbus Drive and Platt Street stations. These time-varying concentrations are shown in Figure 5-6.

Table 5-7 indicates the range of concentrations for various parameters. Nutrient concentrations tend to increase downstream, most likely due to the freshwater release from the Reservoir which flushes out the upstream reaches of the River.

BOD_5 concentrations were low in the River and did not appear to increase significantly after the storm event. Overall TSS levels in the River were low with some increases after the storm event. The concentrations of TSS at Columbus Drive and Platt Street were higher than those at upstream stations.

Coliform bacteria counts were highest in upstream reaches and increased one to two orders of magnitude after the storm. Bacteria counts for both fecal and total coliform remained high for about three days following the storm.

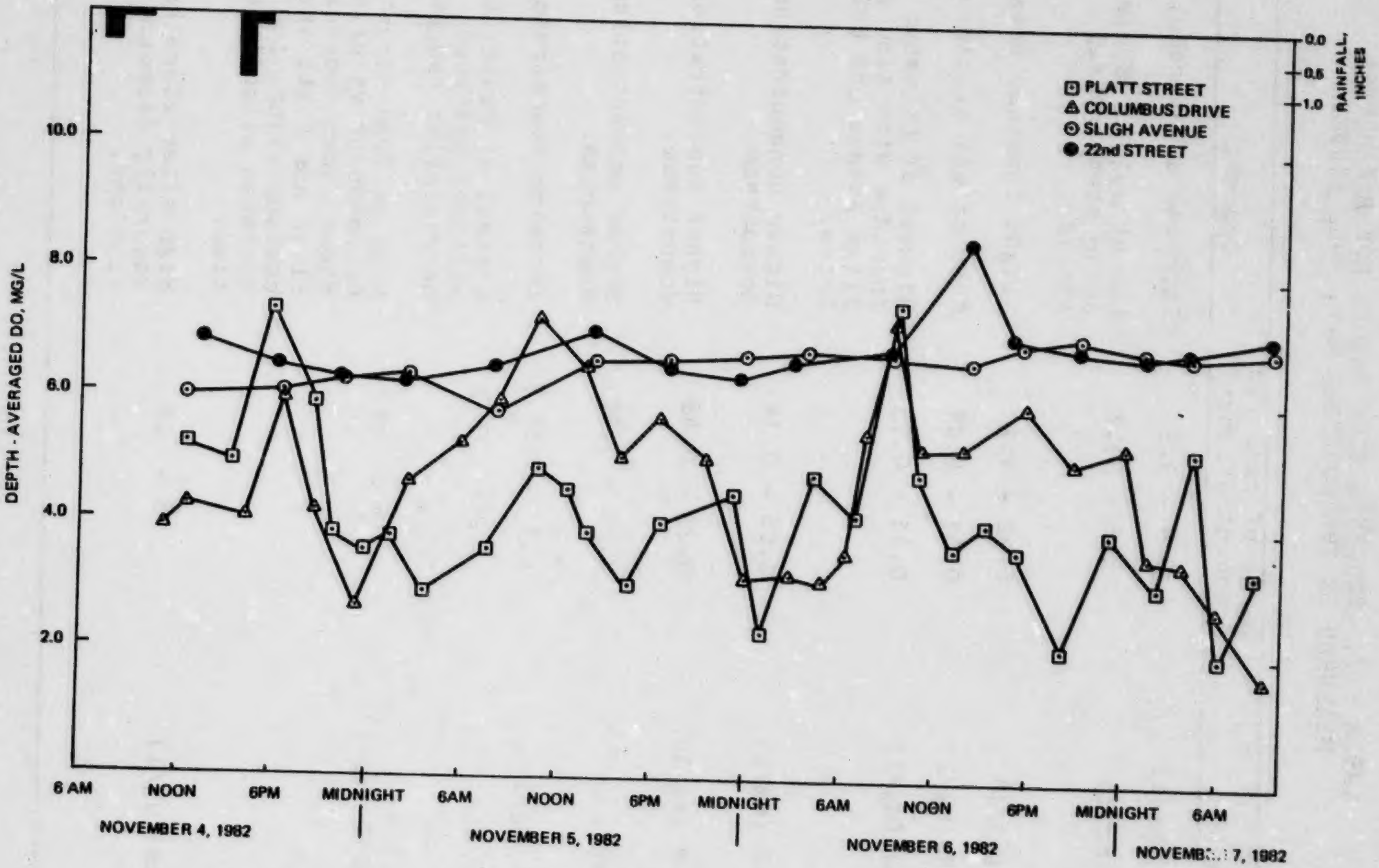


FIG. 5-6 DEPTH-AVERAGED DISSOLVED OXYGEN CONCENTRATIONS - NOVEMBER 4-7, 1982

TABLE 5-7. CONCENTRATION RANGES FOR PARAMETERS MEASURED IN THE NOVEMBER 4-7, 1982 SURVEY

arameter	Range of measured concentrations	Comment
BOD_5 (mg/l)	0.4 - 3.5	Increase downstream.
COD (mg/l)	6.6 - 59.7	Low at Sulphur Springs; other stations all similar.
TKN (mg/l)	0.92 - 1.23	Slight increase downstream.
NO_2-N (mg/l)	0.01 - 0.02	Same at all stations.
NO_3-N (mg/l)	0.11 - 0.73	Highest at Columbus Drive; increase with time at Sligh Avenue and Columbus Drive.
NH_4-N (mg/l)	0.06 - 0.14	Higher concentrations downstream.
PO_4-P (mg/l)	0.23 - 0.48	Higher concentrations downstream.
P (mg/l)	0.26 - 0.52	Higher concentrations downstream.
TSS (mg/l)	<.1 - 87.8	Increase downstream.
VSS (mg/l)	1.2 - 7.5	Highest at Platt; other stations are lower and have similar ranges.
Copper (ug/l)	13 - 57	High at 22nd Street and Columbus Drive after storm, then decreases. At Sligh and Platt intial concentrations low but increase slightly over time.
Lead (ug/l)	8 - 22	High after storm but generally decrease at all stations.

TABLE 5-7 (Continued). CONCENTRATION RANGES FOR PARAMETERS MEASURED IN THE NOVEMBER 4-7, 1982 SURVEY

Parameter	Range of measured concentrations	Comment
Dissolved Lead (ug/l)	4 - 20	In general, high after storm then decrease.
Zinc (ug/l)	3.6 - 22	High after storm then decrease at all stations.
Fecal Coliform (MPN/100 ml)	<100 - 5,300	Higher upstream; greater increase upstream after storm
Total Coliform (MPN/100 ml)	2,600 - 690,000	Higher upstream; greater increase upstream after storm

The range of concentrations of water quality parameters measured during all synoptic surveys is presented in Table 5-8 along with the State of Florida Water Quality Criteria (Chapter 17-3) for Class III waters. The parameters which violate State criteria are dissolved oxygen, fecal and total coliform and copper. The factors which cause DO violations were discussed earlier in this Chapter. Hillsborough Bay water appears to be the primary source of phosphorus in the River, and high coliform and copper concentrations are a result of urban stormwater runoff entering the River.

Vertical Variation

The interaction of the upstream freshwater inflow and downstream tidal variations results in a characteristic estuarine

TABLE 5-8. COMPARISON OF LOWER HILLSBOROUGH RIVER WATER QUALITY CRITERIA WITH RIVER DATA

Parameter	Unit	Criteria	River Data
BOD_5	mg/l	shall not cause DO depressions violating standards	1-16
DO	mg/l	never less than 4 mg/l	0.0-16
COD	mg/l	-	7-106
TSS	mg/l	settle to form putrescent deposits or a nuisance; produce a turbidity nuisance; < 29 NTU's above background; depth of compensation < 10% change	1-71
Ortho-P	mg/l	man-induced nutrient load shall not cause an imbalance in natural populations of aquatic flora and fauna	0.08-0.93
Total P	mg/l		0.15-0.95
Nitrate	mg/l		0.001-0.940
Nitrite	mg/l		0.000-0.022
TKN	mg/l		0.15-5.83
Ammonia-N	mg/l		0.044-0.770
Organic-N	mg/l		0.38-5.60
Total-N	mg/l		0.71-6.10
Fecal Coliform	MPN/100ml	monthly average < 200 daily average < 800	40-26,500
Total Coliform	MPN/100ml	monthly average < 1,000 never > 2,400	300-600,000
Aluminum	ug/l	1,500	50-530
Cadmium	ug/l	5	-
Copper	ug/l	15	3-57
Lead	ug/l	50(1)	4-26
Zinc	ug/l	1,000(1)	4-90

1. Surface Waters: General Criteria

circulation pattern in the lower Hillsborough River. Variations in salinity result from the interaction of freshwater flow at the upstream end, tidal forces at the downstream end, and density difference between the fresh and salt water. The bottom waters tend to flow inward, and the top waters tend to flow outward.

The vertical variations in velocity and salinity (density) also impact the distribution of water quality parameters in the water column. The quality of saline waters intruding upstream in the bottom waters is often different from the freshwater which tends to remain in the upper part of the water column. Other impacts which result in vertical variations of water quality are tidal mixing energy, sediment oxygen demand and sediment resuspension, and surface reaeration.

In Figure 5-7, tide and velocity (surface, mid-depth and bottom) at Platt Street during the February 1983 synoptic survey are shown. During this survey, a large rainstorm occurred at about 4:00 AM on February 2. Before this storm, there was no freshwater flow entering from the Reservoir. The input of flow results in the characteristic estuarine circulation pattern. The surface velocities move mainly downstream, the bottom velocities move upstream, and the mid-depth velocities are in between. In sections where there is no salinity intrusion, there is little vertical variation of the velocity.

The salinities at Platt Street and Sligh Avenue are shown in Figure 5-8 during the February survey. After the influx of

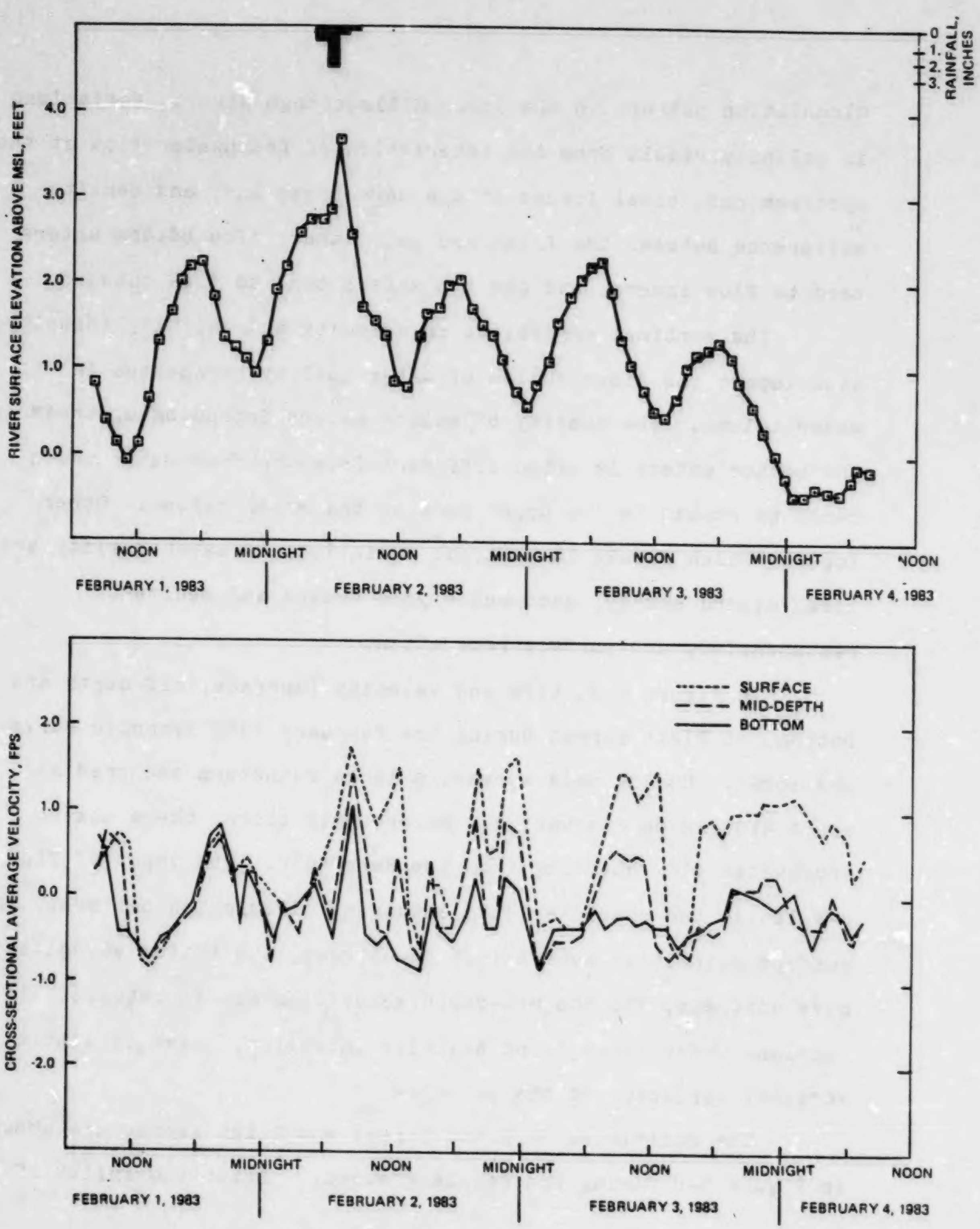


FIG. 5-7 TIDAL ELEVATION AND CURRENT VELOCITY AT PLATT STREET FEBRUARY, 1983

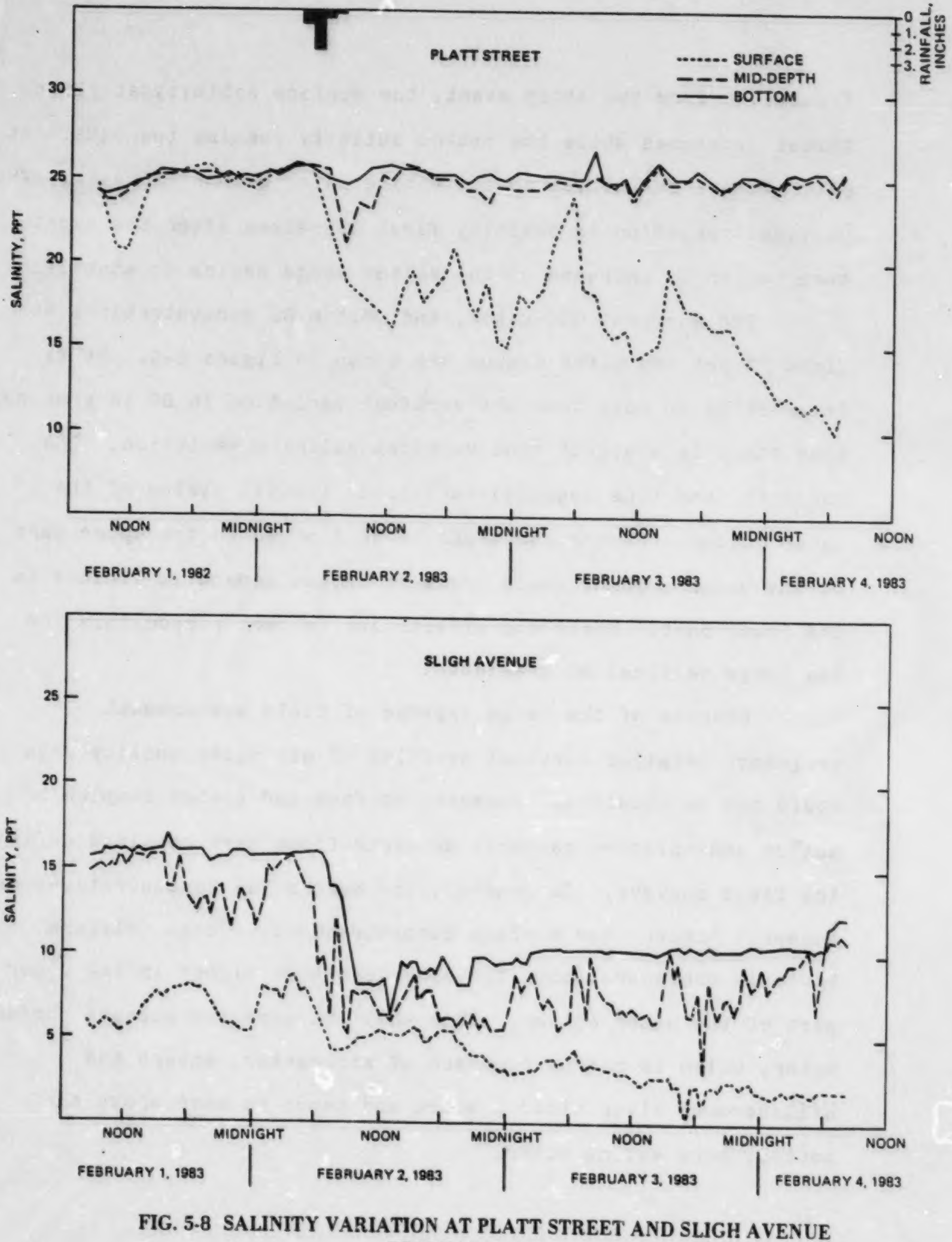


FIG. 5-8 SALINITY VARIATION AT PLATT STREET AND SLIGH AVENUE
FEBRUARY, 1983

freshwater from the storm event, the surface salinity at Platt Street decreases while the bottom salinity remains the same. At Sligh Avenue salinities in the entire water column decrease. The vertical variation in salinity first decreases after the event, then begins to increase as the saline wedge begins to stabilize.

The surface, mid-depth, and bottom DO concentrations at Platt Street and Sligh Avenue are shown in Figure 5-9. It is interesting to note that the vertical variation in DO is greatest when there is a significant vertical salinity variation. The salinity (and thus density) variations inhibit mixing of the water column. Reaeration tends to be limited to the upper part of the water column, while sediment oxygen demand is limited to the lower part. These two effects are in part responsible for the large vertical DO gradients.

Because of the large expense of field measurement programs, detailed vertical profiles of all water quality data could not be obtained. However, surface and bottom samples of solids and coliform bacteria concentrations were obtained during the field surveys. In general, the bottom TSS concentrations are somewhat higher than surface concentrations. Total coliform bacteria concentrations, are also generally higher in the upper part of the water column. This would be expected because surface water, which is mainly composed of stormwater, enters the Hillsborough River after a storm and tends to move above the bottom, more saline water.

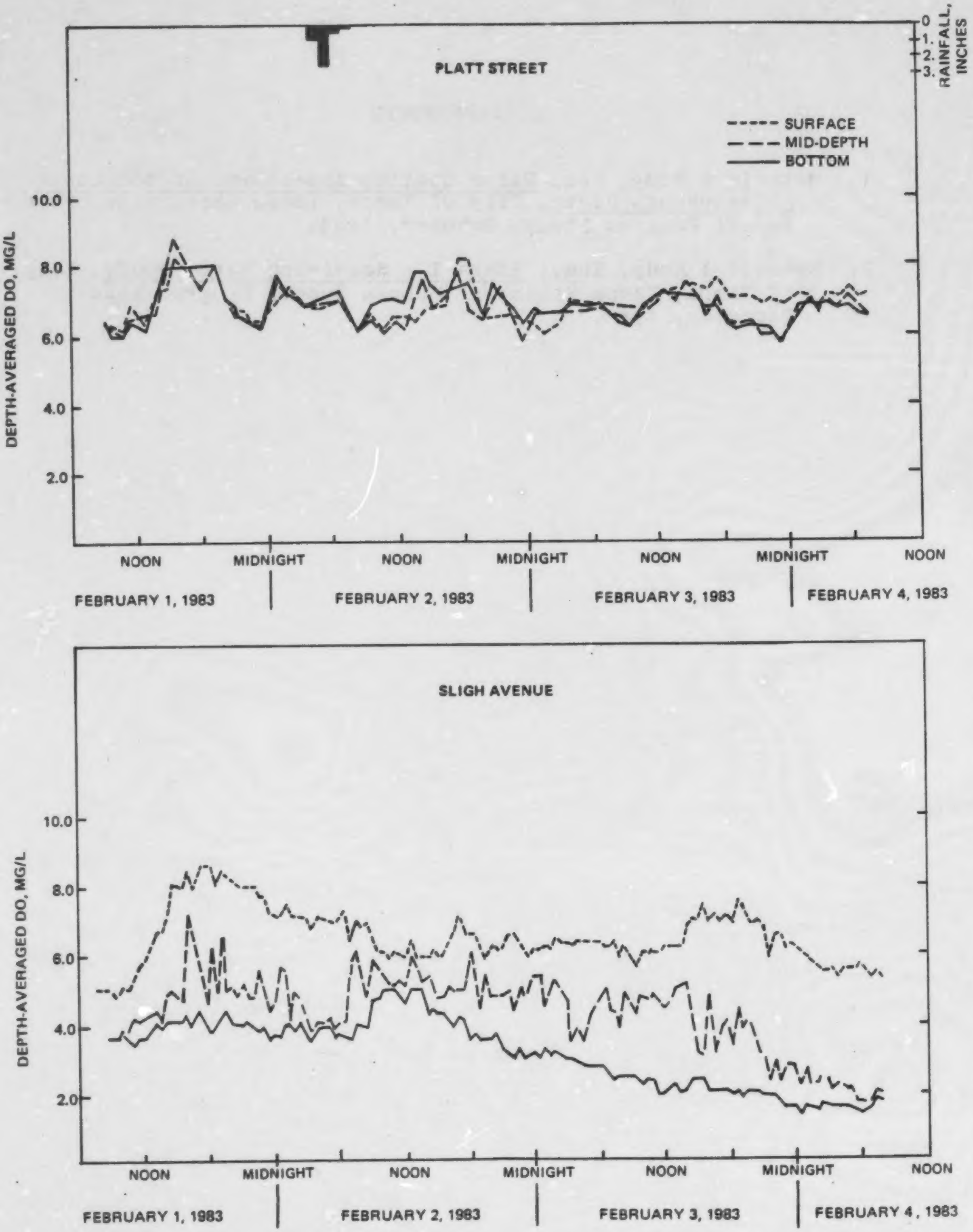


FIG. 5-9 DISSOLVED OXYGEN VARIATION AT PLATT STREET AND SLIGH AVENUE
FEBRUARY, 1983

REFERENCES

1. Metcalf & Eddy, Inc. Water Quality Assessment of the Lower Hillsborough River, City of Tampa, Tampa Nationwide Urban Runoff Program Study, October, 1983.

2. Metcalf & Eddy, Inc., Phase I - Receiving Water Study, City of Tampa, Tampa Nationwide Urban Runoff Program Study, December, 1981.

CHAPTER 6

LOWER HILLSBOROUGH RIVER MODELING

General

The first objective of the Receiving Water Study (1) was to understand the various factors which affect water quality conditions in the lower Hillsborough River and the extent to which they impact the River. A second goal of this study was to predict future water quality conditions based on the implementation of various stormwater management plans in the City of Tampa and other management alternatives for the River.

The continuous data collection program and the synoptic survey program discussed in Chapter 5 provided insight regarding the estuarine behavior of the lower River and the effects of stormwater runoff on hydrodynamics and water quality. In order to predict the impacts of alternative river improvement schemes, a calibrated model capable of simulating hydrodynamics and water quality was required. In this Chapter, the modeling approach, model calibration and verification and assessment of alternatives are presented.

For this study the one-dimensional WQON (Water Quality of Open Channel Networks) model developed at the Massachusetts Institute of Technology (2) was used. WQON is a numerical model which has the capability of predicting hydrodynamics and transient water quality in an estuary. The model respresents the estuary as a one-dimensional (cross-sectionally averaged) system. WQON has the capability of solving for several water quality constituents, including salinity, BOD, DO and coliform bacteria. The numerical solution procedures for WQON are based on the finite element method.

The WQON modeling approach involves three steps as follows:

1. Simulate hydrodynamics (tide and velocity variations).
2. Simulate salinity (and the effect of density variations on hydrodynamics).
3. Simulate water quality.

In the second step, where salinities are simulated, the hydrodynamic results are modified because of a coupling in the model between hydrodynamics and salinity. This coupling accounts for the effect of salinity (and thus density) variations on the cross-sectionally averaged velocities. Once these steps are completed, water quality parameters such as BOD and DO may be simulated.

The key model inputs for the lower Hillsborough River analysis include physical system characteristics, tides, freshwater flows, water quality boundary conditions, and reaction rates. The City of Tampa provided detailed data on 21 cross-sections along the lower Hillsborough River as well as a bottom profile of the River obtained from a fathometer run.

The Platt Street station (Figure 5-1) was selected as the downstream or Bay boundary. Hydrodynamics and water quality data from the synoptic surveys were used for specifying boundary conditions for model runs. The Tampa Dam was used as the upstream boundary for model runs. Varying freshwater discharges from the Hillsborough Reservoir as measured by the U.S.G.S. and concentrations of water quality constituents measured at 22nd Street (Figure 5-1) were specified at this boundary.

Longitudinal dispersion was accounted for using empirical formulas presented by Thatcher et al (3). Estimates of dispersion in the saltwater and freshwater zones were calculated based on stream velocity, Mannings' n, the estuary length and the estuary number.

First order reaction rates are assumed for all constituents solved by WQON and these rates were input to the model. A BOD decay rate of 0.23/day, base e, was assumed as a typical decay rate for such systems (no long-term BOD tests were performed on River samples). Dissolved oxygen reaeration coefficients were calculated using the O'Connor-Dobbins formula which was most applicable to characteristic depths and velocities in the lower Hillsborough River. Sediment oxygen demand rates as measured from field experiments (4,5) were input as oxygen sinks along the River.

Model Calibration

To calibrate the model for River hydrodynamics, the measured data from the April 29-30, 1982 synoptic survey were used. The model inputs are summarized in Table 6-1. Figure 6-1 presents both measured and model predicted river surface elevations and stream velocities at Sligh Avenue. Positive

TABLE 6-1. WQON MODEL INPUTS FOR CALIBRATION RUN

Parameter	Model Input
Tidal Variation at Bay Boundary	As measured at Platt Street on April 29-30, 1982
Freshwater Flow from Reservoir	119-124 cfs as measured by USGS
Sulphur Springs Discharge	48 cfs
Salinity at Bay Boundary	24.8 ppt
Salinity at Upstream Boundary	0.0 ppt
Dispersion Parameter, K	160 ft^2/sec
BOD at Bay Boundary	As measured at Platt Street on April 29-30, 1982
BOD at Upstream Boundary	As measured at 22nd Street on April 29-30, 1982
DO at Bay Boundary	As measured at Platt Street on April 29-30, 1982
DO at Upstream Boundary	As measured at 22nd Street on April 29-30, 1982
BOD Decay Rate	0.23/day
DO Reaeration Rate	0.17 - 0.32/day
Sediment Oxygen Demand	2.6 - 9.8 $gO_2/m^2/day$

velocities indicate downstream flow and negative velocities represent upstream flow.

To reproduce salinity conditions in the River, the extent of saline penetration upriver, the Reservoir freshwater release and the variation of salinity concentration along the River are taken into account. Figure 6-2 presents the measured and predicted salinity concentrations at Platt Street and Columbus Drive.

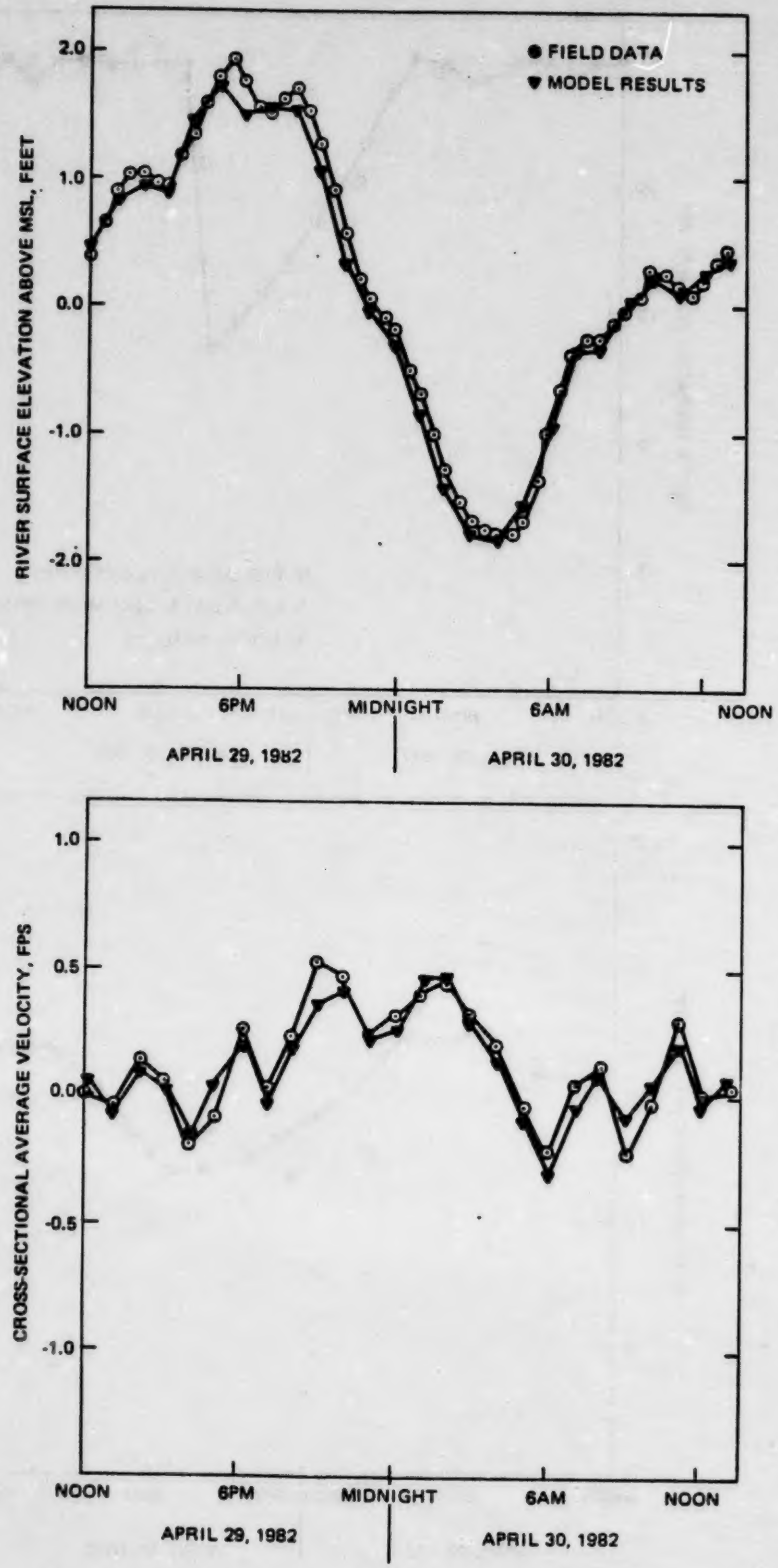


FIG. 6-1 HYDRODYNAMIC CALIBRATION RESULTS AT SLIGH AVENUE

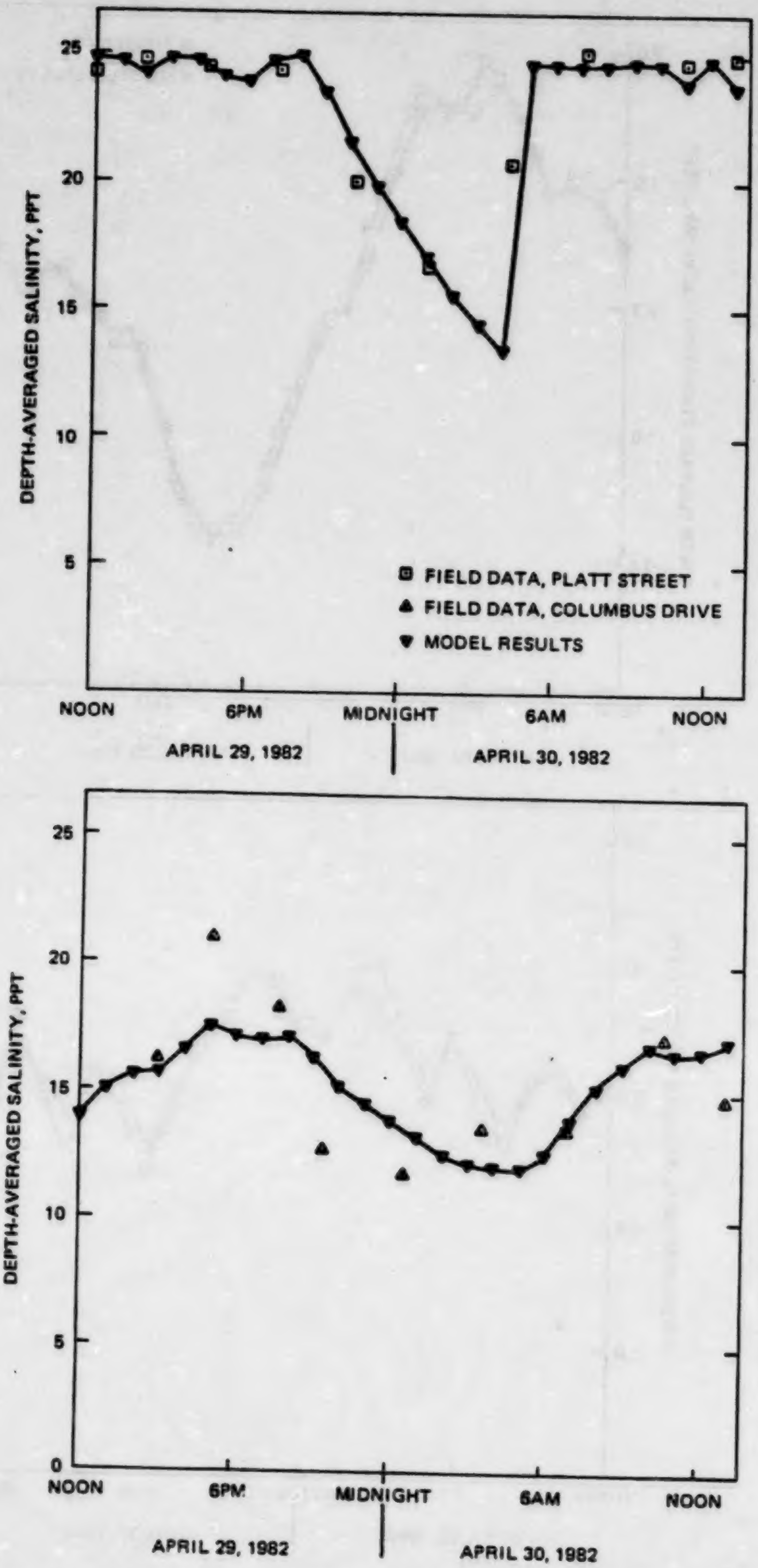


FIG. 6-2 SALINITY CALIBRATION RESULTS AT PLATT STREET AND COLUMBUS DRIVE

Model calibration for BOD and DO variations in the lower Hillsborough River was also performed. Figure 6-3 shows an example of model-predicted BOD and DO concentrations and measured data at Sligh Avenue.

Model Verification

The November 1982 survey was used for the first model verification. A summary of model inputs is presented in Table 6-2. The storm event that occurred near the start of this survey was input to the S/SWMM model and routed through all the drainage basins. The outflows from all the basins were input as lateral inflows to the lower Hillsborough River in the WQON model. An example of the hydrodynamic results is presented in Figure 6-4. An excellent match is observed for gage height and stream velocities at Sligh Avenue.

The results of salinity verification runs are presented in Figure 6-5. The model predictions and field data measurements show good agreement at both stations.

BOD and DO concentrations were also simulated by the model for the November survey. A BOD_5 loading of 3.8 mg/l in the drainage areas, based on runoff concentrations observed during the Phase II study on areawide pollutant loadings (6), appeared to agree well with the measured River data as seen in Figure 6-6. This indicates that the storm event occurring during this survey did not contribute large amounts of BOD to the River. The predicted and measured DO concentrations at Sligh Avenue are also presented in Figures 6-6.

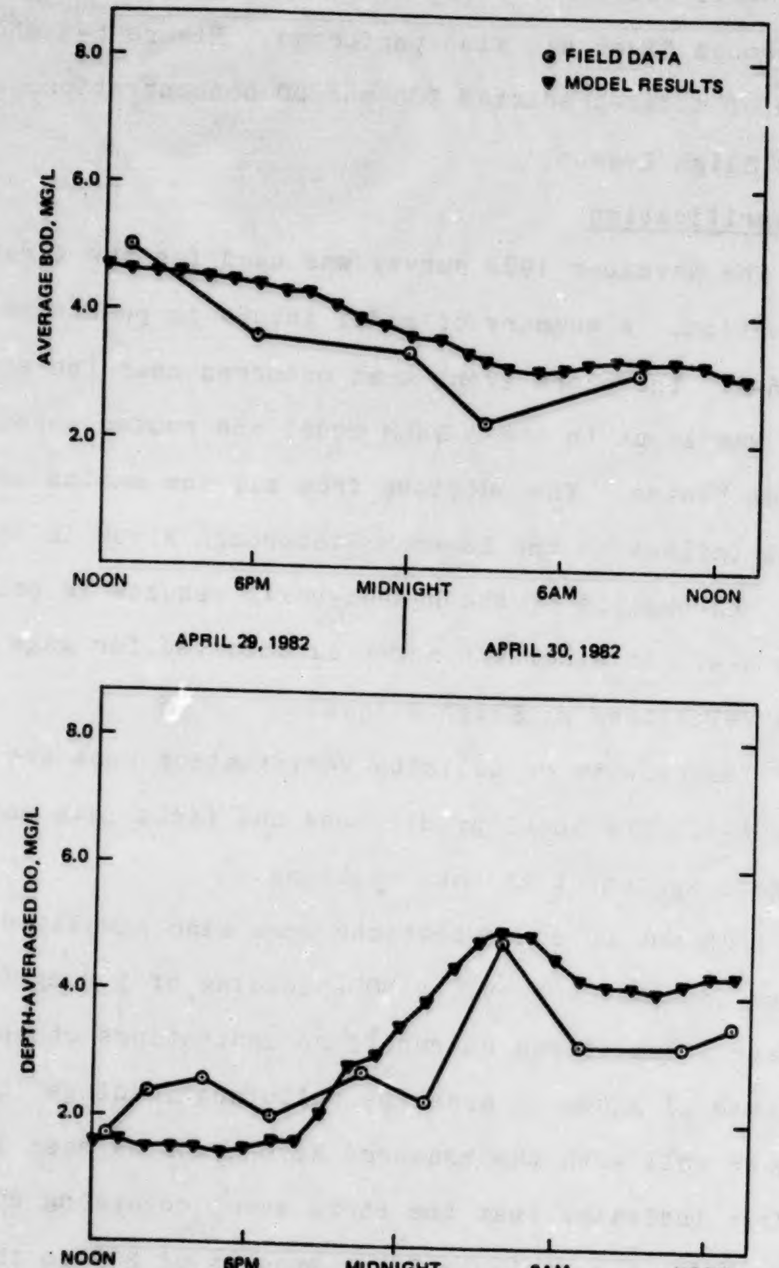


FIG. 6-3 BOD AND DO CALIBRATION RESULTS AT SLIGH AVENUE

TABLE 6-2. WQON MODEL INPUTS FOR FIRST VERIFICATION RUN

Parameter	Model Input
Tidal Variation at Bay Boundary	As measured at Platt Street on November 4-7, 1982.
Freshwater Flow from Reservoir	290-510 cfs as measured by USGS
Sulphur Springs Discharge	41.5 cfs
Salinity at Bay Boundary	20.0 ppt
Salinity at Upstream Boundary	0.0 ppt
Dispersion Parameter, K	228 ft^2/sec
BOD at Bay Boundary	As measured at Platt Street on November 4-7, 1982
BOD at Upstream Boundary	1.5 mg/l
BOD Loading in Stormwater Runoff	3.8 mg/l
DO at Bay Boundary	As measured at Platt Street on November 4-7, 1982
DO at Upstream Boundary	7.0 mg/l
DO of Stormwater Runoff	5.0 mg/l
BOD Decay Rate	0.23/day
DO Reaeration Rate	0.13 - 0.55/day
Sediment Oxygen Demand	1.1 - 3.6 gO_2/m^2/day

A second model verification was performed using data collected in the February 1983 survey. A large storm occurred about 20 hours after the survey began; therefore, pre-storm water quality conditions were clearly defined in the data. This enabled the calibration of fecal coliform in the River due to a

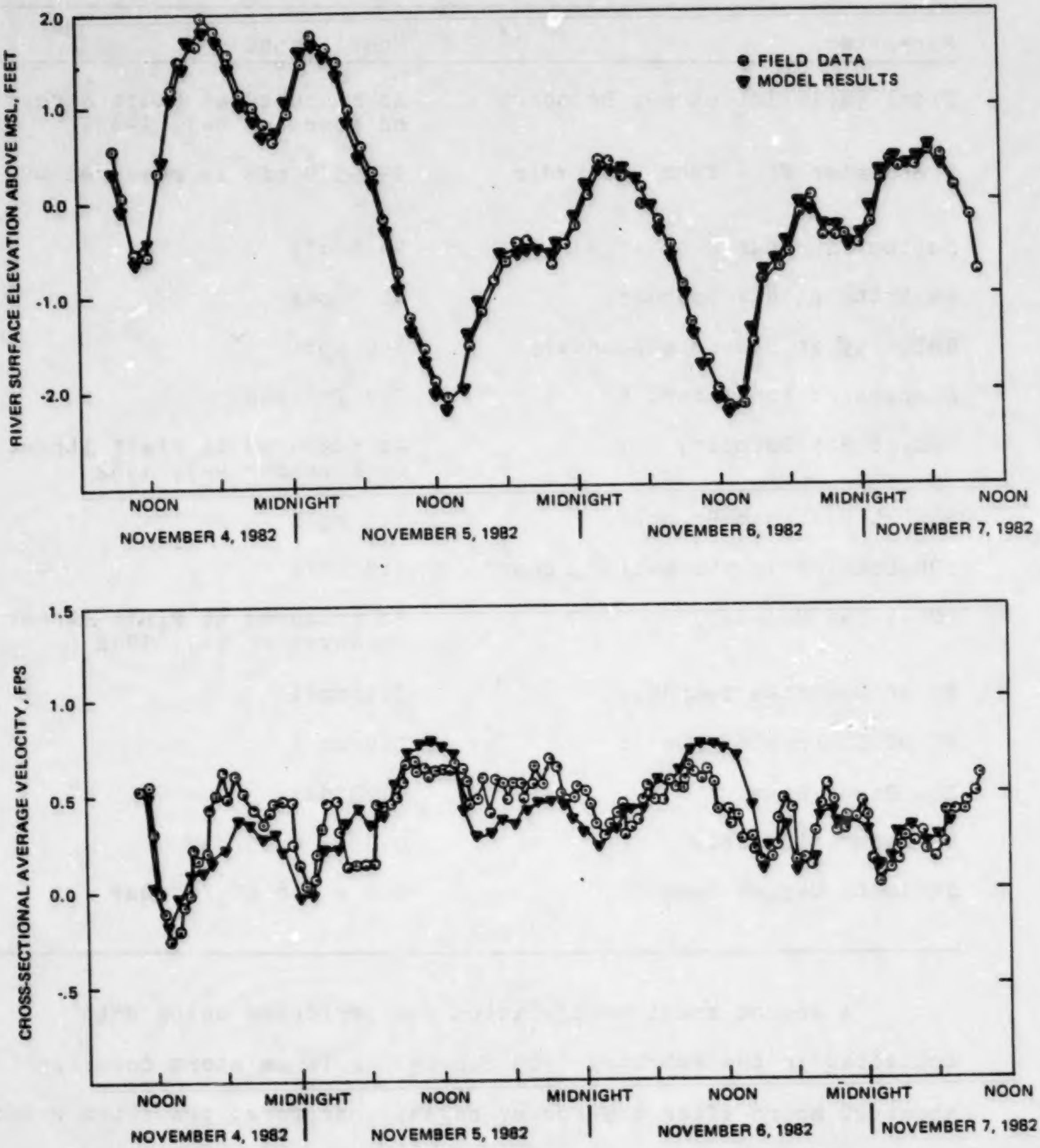


FIG. 6-4 HYDRODYNAMICS AT SLIGH AVENUE - FIRST VERIFICATION

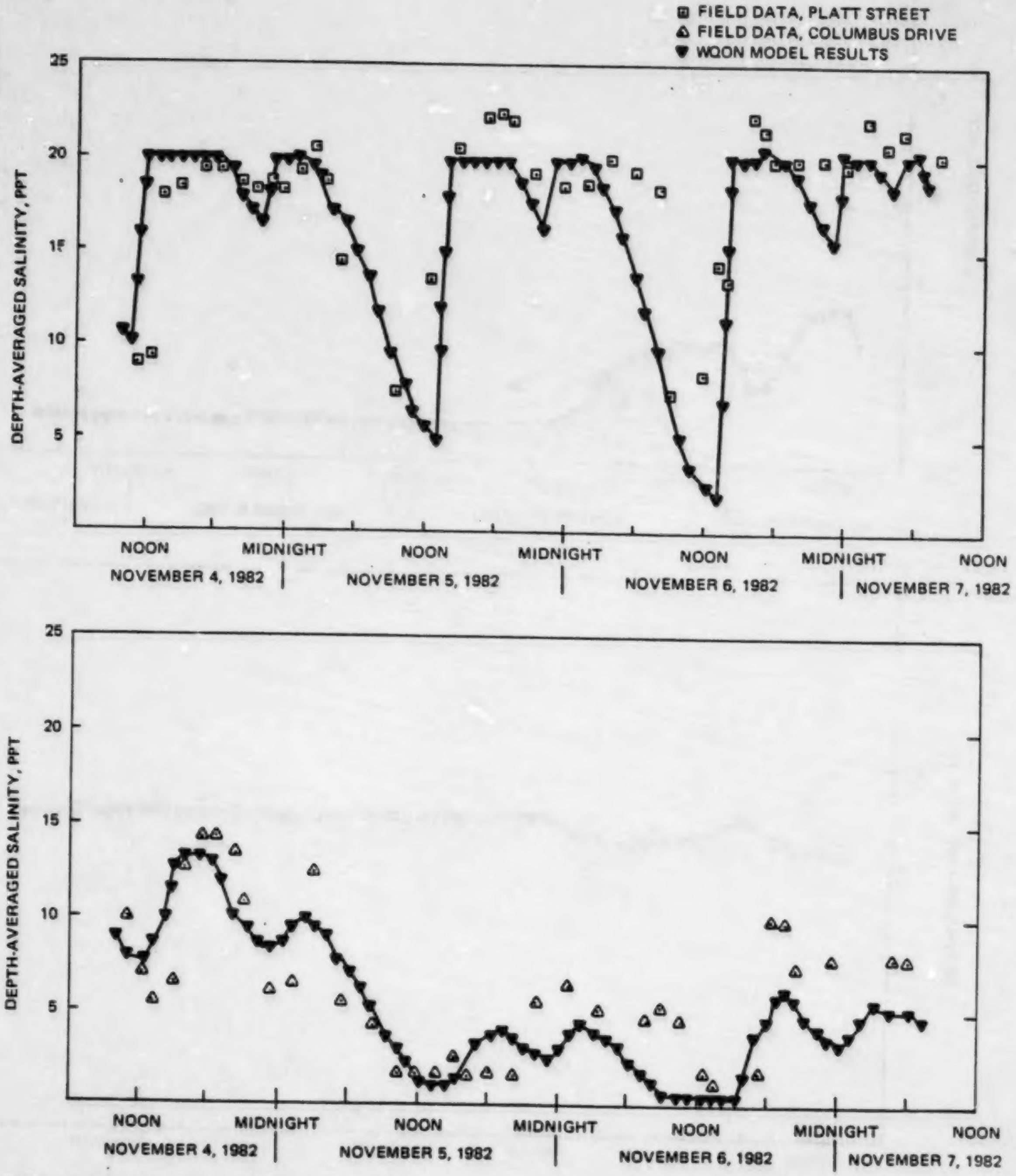


FIG. 6-5 SALINITY CONCENTRATIONS AT PLATT STREET AND COLUMBUS DRIVE
FIRST VERIFICATION

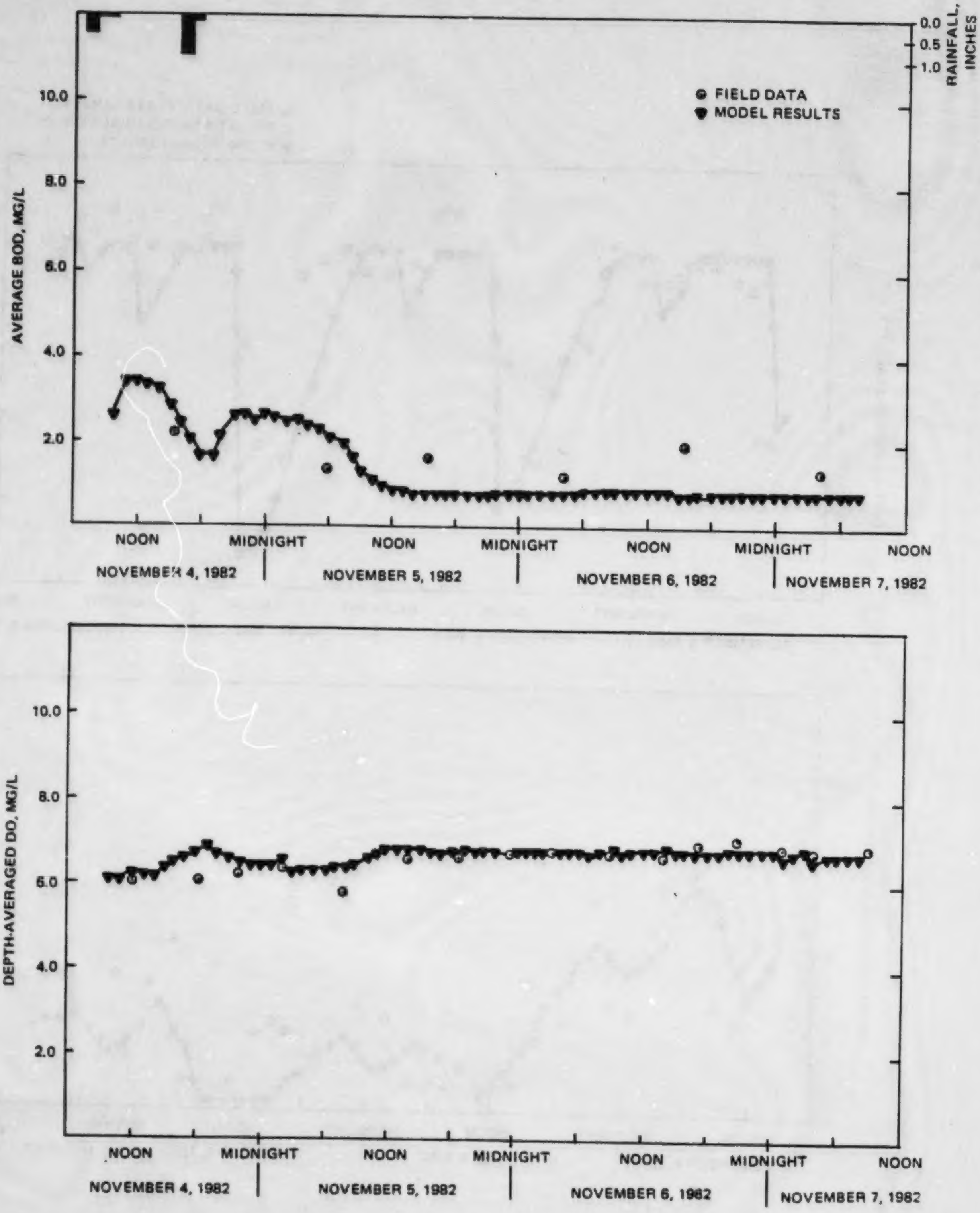


FIG. 6-6 BOD AND DO CONCENTRATIONS AT SLIGH AVENUE
FIRST VERIFICATION

storm event. Fecal coliform concentrations of 25,400 MPN/100 ml were assumed for stormwater runoff(6). A decay rate of 2.0/day was found to provide the best agreement of model predictions with field data. The model predictions of coliform concentrations at Columbus Drive and Sligh Avenue are presented in Figure 6-7. Again, model results reflect very good agreement with field data.

Analysis of Vertical Salinity and DO Profiles

To take into account the effect of observed vertical variations in field data, an analysis of the vertical salinity profiles was performed. Dimensionless plots were prepared at each station for salinity versus depth. Plots were prepared at mid-ebb, high, mid-flood, and low tides.

Dimensionless plots of salinity versus depth at Columbus Drive are presented in Figures 6-8. These figures clearly portray the variation in salinity concentrations over the tidal cycle under various freshwater flow conditions. Strong vertical salinity gradients are observed most clearly for flows of 270 and 1000 cfs. For lower freshwater flows, the Columbus Drive station is nearly all saline. Such dimensionless plots were also prepared for 22nd Street, Sligh Avenue, and Platt Street.

The dimensionless plots of salinity can be used to directly predict vertical variation of salinity at Columbus Drive or the other stations for any freshwater flow conditions. The procedure is to take the cross-sectional average value predicted by WQON and back-calculate the vertical profile from the dimensionless plot for the specified flow and tidal time.

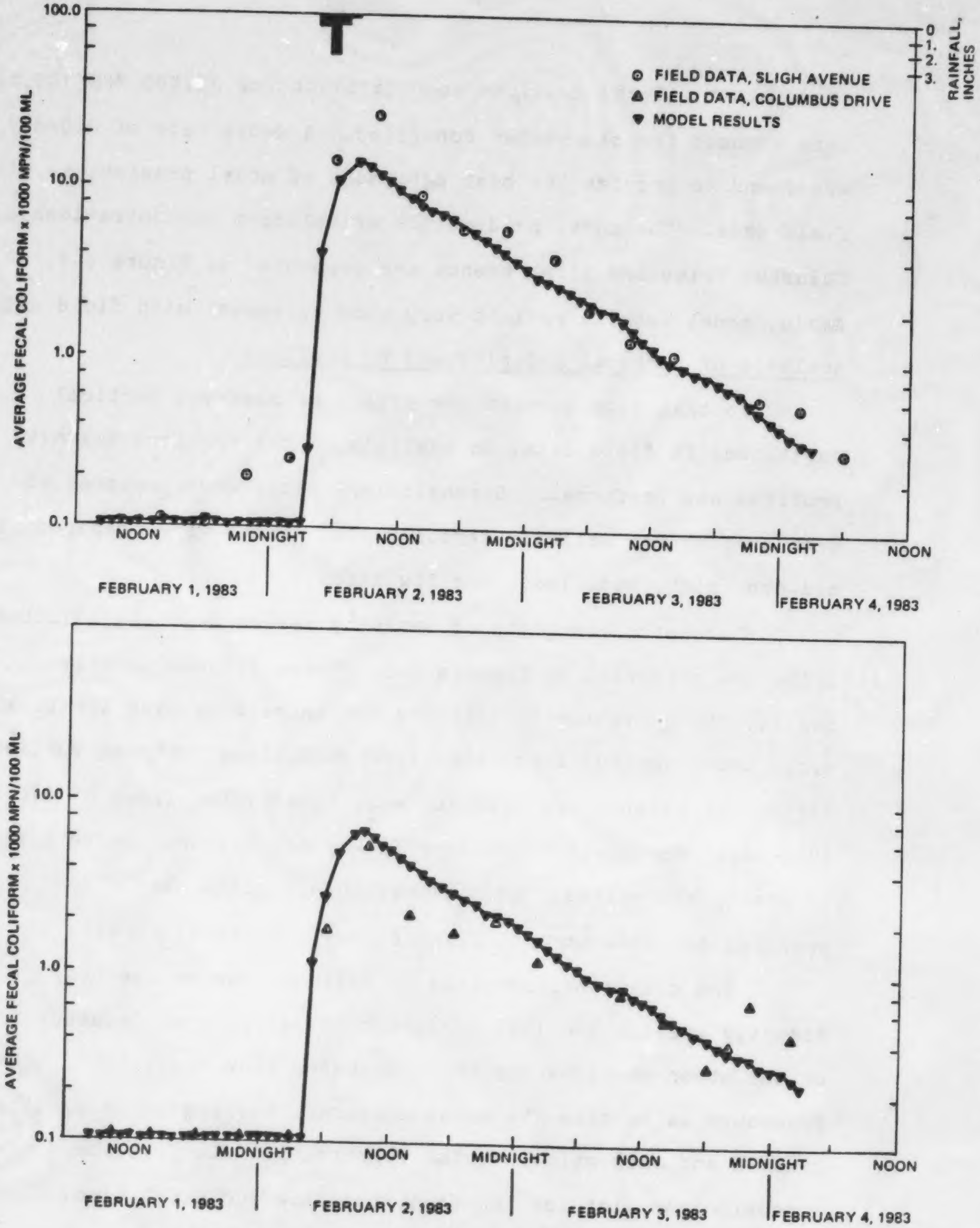


FIG. 6-7 FECAL COLIFORM CALIBRATION RESULTS AT SLIGH AVENUE AND COLUMBUS DRIVE

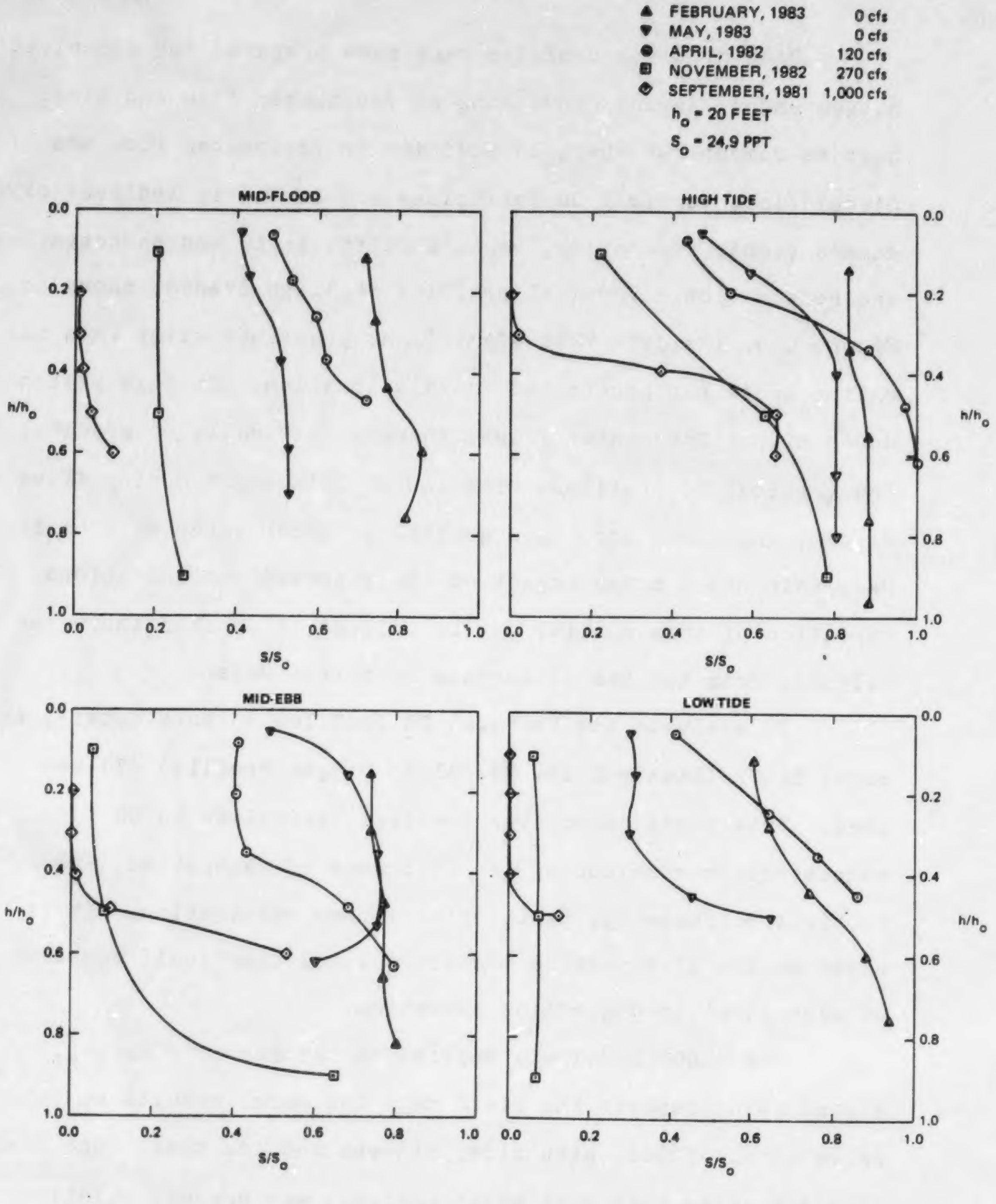


FIG. 6-8 DIMENSIONLESS SALINITY PROFILES AT COLUMBUS DRIVE FOR VARIOUS FRESHWATER FLOW CONDITIONS

Dimensionless profiles were also prepared for dissolved oxygen under varying conditions of freshwater flow and tide. It must be remembered that, in addition to freshwater flow and tidal circulation, vertical DO variations are caused by sediment oxygen demand (SOD), reaeration, vertical diffusivity and photosynthesis and respiration. Vertical profiles at Sligh Avenue, shown in Figure 6-9, indicate that significant gradients exist when the saline wedge has penetrated to this location. At this station, under higher freshwater flows, there is virtually no gradient in the vertical DO profiles. The lowest DO's occur during flows of 0, 100, and 1,000 cfs. The quality of water entering from the Reservoir has a major impact on the observed concentrations. The variation of this quality may be related to whether the water released from the Dam is surface or bottom water.

To evaluate the vertical DO profiles in more detail, the model SSDOP (Steady State Dissolved Oxygen Profile) (7) was used. This model calculates vertical variations in DO concentration considering the influence of reaeration, SOD, vertical diffusivity, photosynthesis and respiration. It is based on the steady-state one-dimensional (vertical) conservation of mass equation neglecting advection.

The SSDOP Model was applied to the February survey. Figure 6-10 presents the field data and model results at Columbus Drive at mid-flood, high tide, mid-ebb and low tide. The field data indicated that some algal activity was present at all stations during this survey. The rate of photosynthesis was

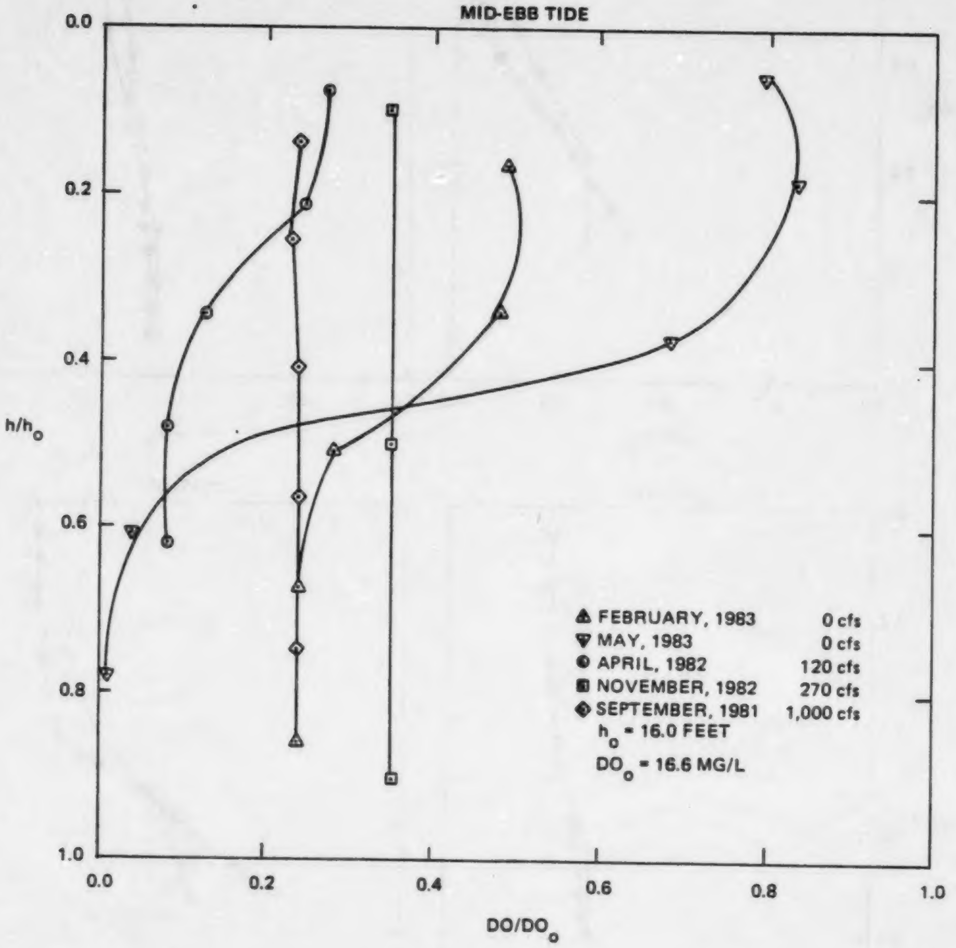


FIG. 6-9 DIMENSIONLESS DO PROFILES AT SLIGH AVENUE FOR VARIOUS FRESHWATER FLOW CONDITIONS

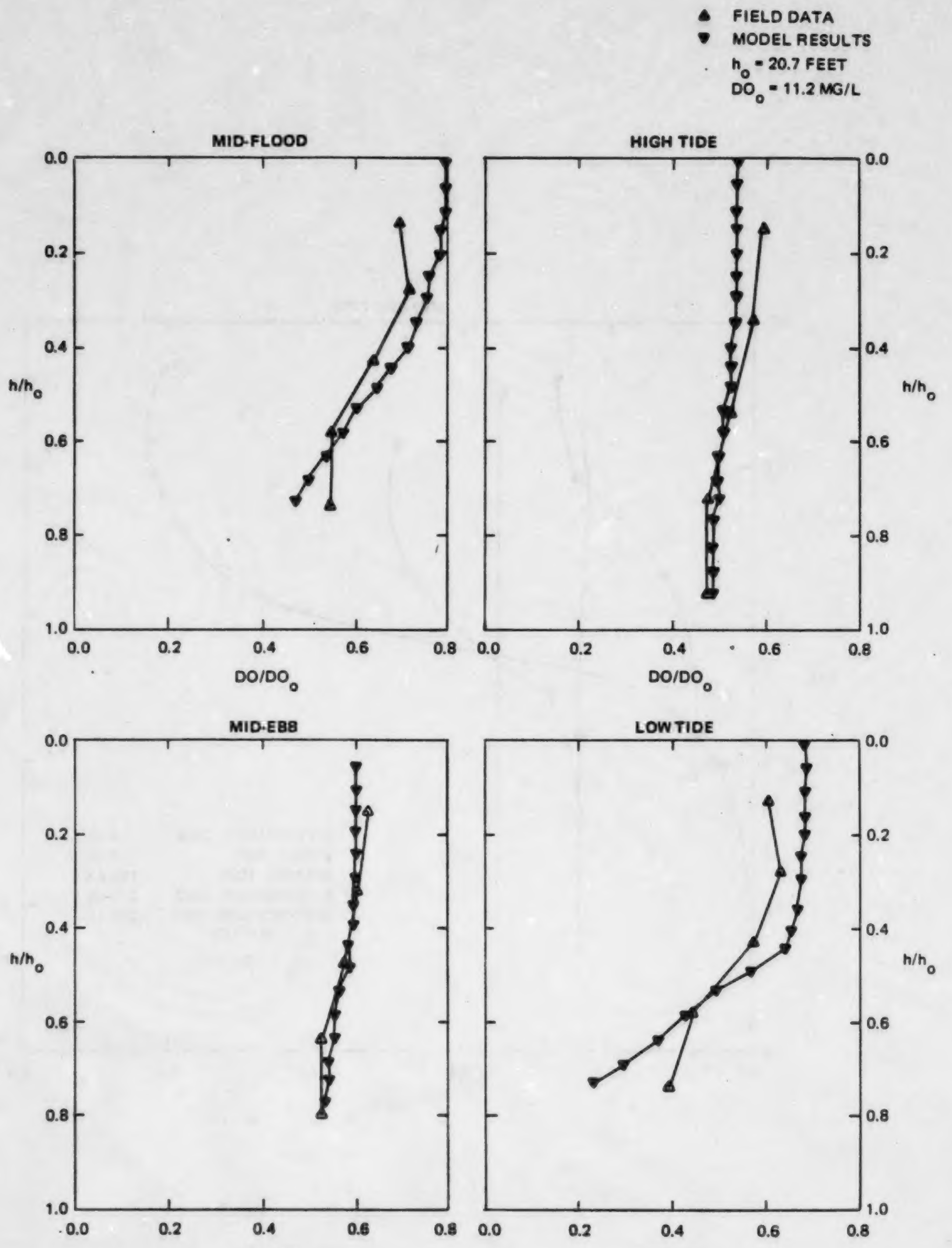


FIG. 6-10 SSDOP MODEL RESULTS AT COLUMBUS DRIVE - FEBRUARY, 1983

therefore adjusted until a good fit was observed between SSDOP results and field data.

The SSDOP model applies to segments of the River that are vertically stratified. The WQON results can be assumed to represent DO concentrations for the entire water column for segments of the River that are not stratified.

Assessment of Alternatives

The water quality problems of the Hillsborough River are due to many interactive factors. Several alternatives have been developed to address the concerns of low dissolved oxygen concentrations, high coliform bacteria levels and nutrient concentrations in the River. Each alternative is designed to counteract the negative impacts on water quality due to one major source. The alternatives are presented as follows.

Salt Water Barrier. A salt water barrier erected at the mouth of the lower Hillsborough River would prevent Hillsborough Bay water from transporting and depositing organic solids upstream. The elimination of salinity would improve dissolved oxygen concentrations in the River. Nutrients would not enter the River from the Bay and sediment accumulation on the River bottom would decrease. A salt water barrier, however, may affect the upstream migration of salt water species and recreational boating activities.

Regulation of Reservoir Releases. As discussed in Chapter 2, the dissolved oxygen concentration in the lower Hillsborough River are dependent on the freshwater flows from

Hillsborough Reservoir and the resulting location of the saline wedge. If the Reservoir releases could be regulated so as to assure a continual freshwater flow to the River, improvements in dissolved oxygen concentrations would be observed along the River. The length of the River in which DO improvements would result would depend on the magnitude of the freshwater release. Table 6-3 estimates the extent of average DO levels above 5.0 mg/l for various continual releases from the Reservoir.

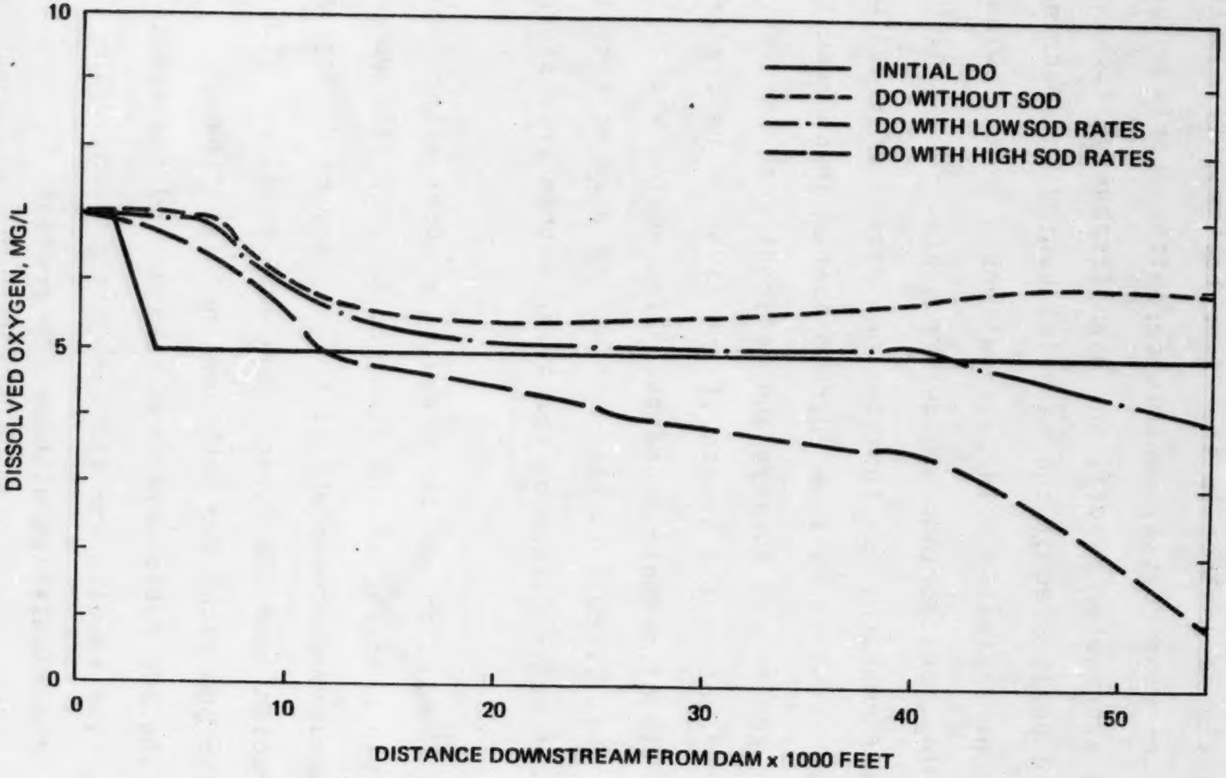
TABLE 6-3. IMPROVEMENTS IN DO CONCENTRATIONS AND NECESSARY FRESHWATER RELEASES

Extent of river with DO >5.0 mg/l	Continuous reservoir release (cfs)
Tampa Dam to 22nd Street	50
Tampa Dam to Sligh Avenue	200
Tampa Dam to Columbus Drive	1,000
Tampa Dam to Platt Street	>1,500

With the existence of a salt water barrier at the mouth of the River, a continuous Reservoir release would be desirable to prevent stagnant conditions in the River; however, high discharges would not be necessary since salinity intrusion would be nonexistent.

Dredging. The downstream segments of the River bottom are covered with thick organic muds that exert a significant oxygen demand on the water column. Dredging of the benthic deposits would eliminate the oxygen demand for a time, but it is likely that material from Hillsborough Bay would continue to be transported and deposited upstream and solids from stormwater runoff would again accumulate in the River bottom. If a salt water barrier was constructed, benthic deposition would occur primarily from stormwater runoff, and high freshwater flows would transport and deposit most of the material behind the barrier. Unless dredged periodically this material could create nuisance conditions during long periods of low freshwater releases. Based on available information, the longitudinal extent of dredging activity is estimated to be from Buffalo Avenue downstream to Platt Street. Additional surveys and sediment core samples would be necessary to confirm the length of the River to be dredged as well as the depth of organic deposits. If dredging was performed, it is difficult to estimate at the present time the rate of sediment accumulation on the River bottom from stormwater runoff alone.

A WQON simulation was performed to predict water quality conditions after dredging of the River bottom. A salt water barrier was assumed at the mouth of the estuary resulting in no salinity intrusion into the River. The WQON model was run for eight days with SOD rates for both low and high flow conditions. The SOD rates were then removed and the simulation was repeated. The results for all runs are presented in Figure 6-11. The simulation without SOD predicts DO concentrations between 5 and 7 mg/l. The simulations with SOD



NOTE: SALTWATER BARRIER
ASSUMED AT RIVER MOUTH
RESERVOIR RELEASE 50 cfs
SULPHUR SPRINGS 40 cfs

FIG. 6-11 DISSOLVED OXYGEN PROFILES FOR A RIVER SIMULATION WITH AND WITHOUT SEDIMENT OXYGEN DEMAND

rates, however, show a decrease in dissolved oxygen concentrations in the lower segments of the River. DO concentrations with SOD are up to 5 mg/l less than predicted concentrations without SOD.

Disinfection. Coliform counts are very high in the lower Hillsborough River for at least three days after most storm events. Furthermore, measurements of coliform during dry weather indicate that other sources of coliform in the River are still active and result in frequent violations of State Water Quality Criteria.

In order to reduce coliform bacteria levels in the River to within the Standards criteria, the dry weather sources of coliform have to be investigated and controlled first. To prevent violations of coliform counts due to stormwater runoff, control facilities would have to be built to intercept and disinfect stormwater runoff before it enters the River.

These control facilties would create significant capital as well as annual O&M costs. However, the reduction in coliform counts has to be on the order of at least 90 percent of the inflow concentration in order to achieve desired concentrations in the River.

Stormwater Management Plan Alternative. The Stormwater Management Plan for the City of Tampa as presented in the Phase III Report (8) is aimed at reducing stormwater pollutant loads to the maximum extent possible. The greatest potential for this reduction lies in the enforcement of the Stormwater Permitting Rule which requires new developments within the City to either retain, or detain and filter, the runoff generated from one inch or more of rainfall.

As presented in the Phase III Report, flood mitigation by runoff reduction measures could result in stormwater runoff reductions of up to 50 percent in the overall study area. Since other flood mitigation measures will probably be used in conjunction with runoff reduction measures, a conservative stormwater load reduction of 30 percent was assumed for a demonstration of River water quality improvements. The design storm discussed earlier was simulated again with reduced loads. Figure 6-12 shows the initial DO concentrations and the predicted concentrations for existing and future (30 percent removal) conditions 18 hours after the storm event. A DO sag still occurs along the River in the future simulation, but DO improvements of 0.2 to 0.5 mg/l are observed.

Summary

A complex interaction of tides, freshwater flows, and meteorological conditions affects the water quality of the lower Hillsborough River. In general, water quality conditions can improve or degenerate at various locations along the River based on the change in any one of the governing factors. In the same light, long-term water quality improvements in the lower Hillsborough River cannot be realized with the reduction or alteration of only one factor that affects water quality. Rather, an interaction of management alternatives will be necessary to best achieve and maintain the desired water quality conditions in this River.

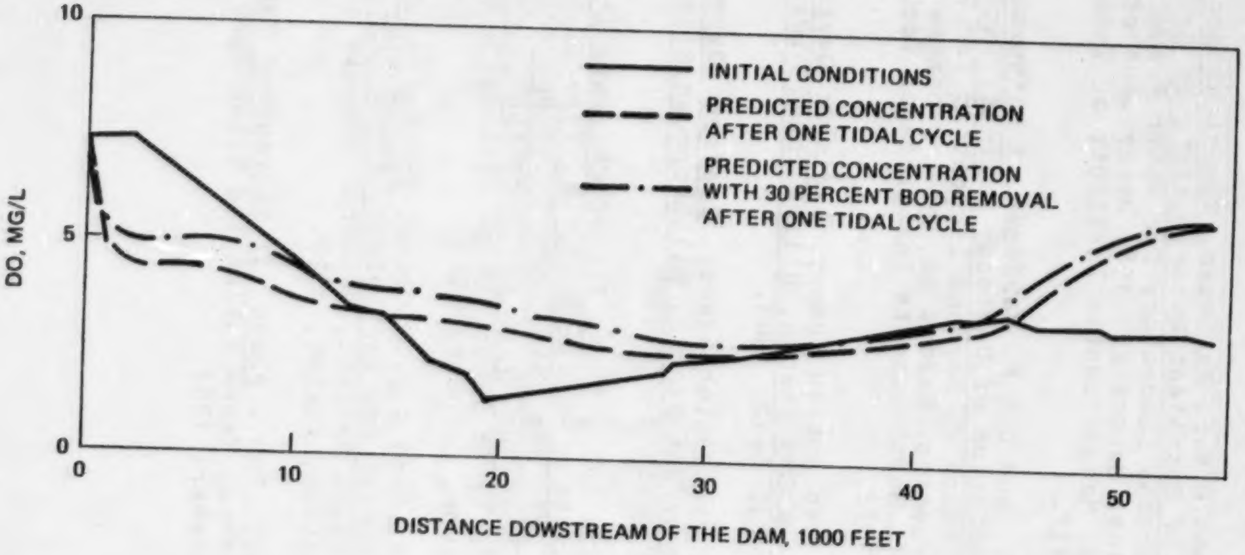


FIG. 6-12 EXISTING AND FUTURE (30 PERCENT BOD REMOVAL) DO PROFILES FOR DESIGN STORM SIMULATION

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CHAPTER 7

STORMWATER MANAGEMENT PLAN

General

In this Chapter, the Stormwater Management Plan (SMP), which was developed to provide the City with integrated stormwater management practices that reflect the scope of the flooding and pollution problems associated with stormwater and current stormwater regulation, is described.

Specific issues addressed in these areas are characterization of the reductions in flows and loads required from the overall study area to eliminate flooding and attainment of receiving water quality objectives. First, stormwater management of flooding is considered. Then, stormwater quality considerations are introduced. Both quantity and quality management recommendations are interpreted with respect to State of Florida permitting specifications. Finally, the integrated approach is demonstrated in a test case area.

Stormwater Quantity Management

The first step in development of a Stormwater Management Plan (SMP) for the City of Tampa was to address the quantity problems associated with stormwater. In this section, existing flooding in the lower Hillsborough River drainage basin and flood mitigation methods are described.

Design Conditions. Design storms with return periods of five, twenty-five and fifty years were developed from the available rainfall record described in Chapter 4. Storm magnitudes, intensities and durations were determined for each design storm by analyzing this record. The intensity-duration-frequency curves derived from the record are shown in Figure 7-1. Included in this Figure for comparative purposes is the Florida Department of Transportation (FDOT) five-year curve for Zone 6, Tampa, currently used by the City. An example hyetograph subsequently developed from the curves derived from the airport records is shown in Figure 7-2.

Design Storm Flooding. The existing storm sewer system with land use as projected by the City for the year 2000 (1) was defined as the future no-action condition for flood assessment. Flooding was first assessed under this condition in order to ascertain changes needed to prevent future flooding.

The three design storms were routed through the lower Hillsborough River basin using the S/SWMM. The response of the basin to these storms was then analyzed to identify problem subareas within the basin.

Table 7-1 characterizes flooding in the overall study area for the three design storms under the no-action condition. As seen from Table 7-1, the existing storm sewer system does not provide adequate protection even for the five-year eight-hour storm. Flooding is significant during all three design storms with at least 59 percent of the subareas experiencing some flooding. Extensive drainage network design changes will be required to prevent flooding for these storms. Guidelines were

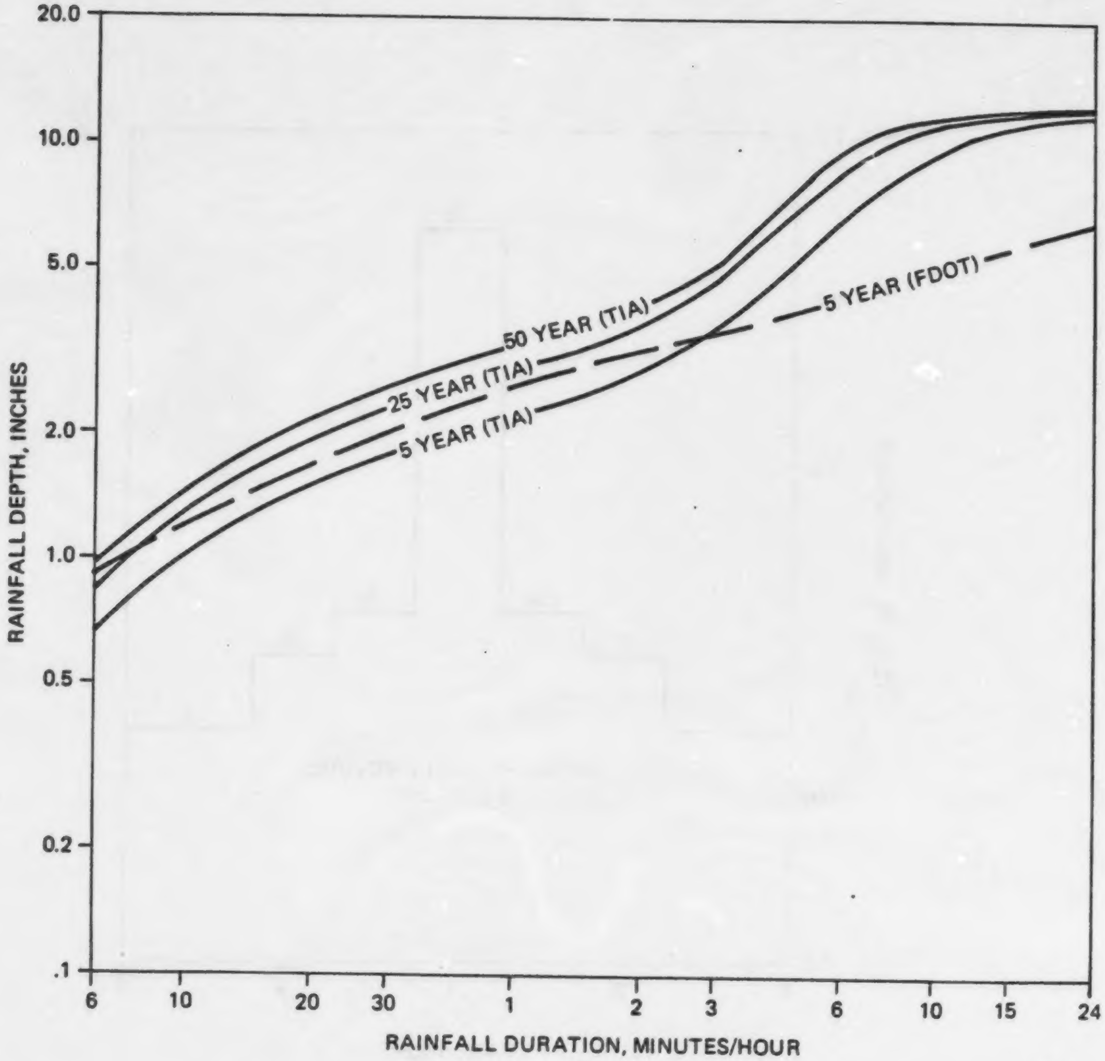

FIG. 7-1 RAINFALL INTENSITY-DURATION-FREQUENCY CURVES FROM TAMPA INTERNATIONAL AIRPORT (TIA) RAINFALL RECORDS AND FROM FDOT DRAINAGE MANUAL

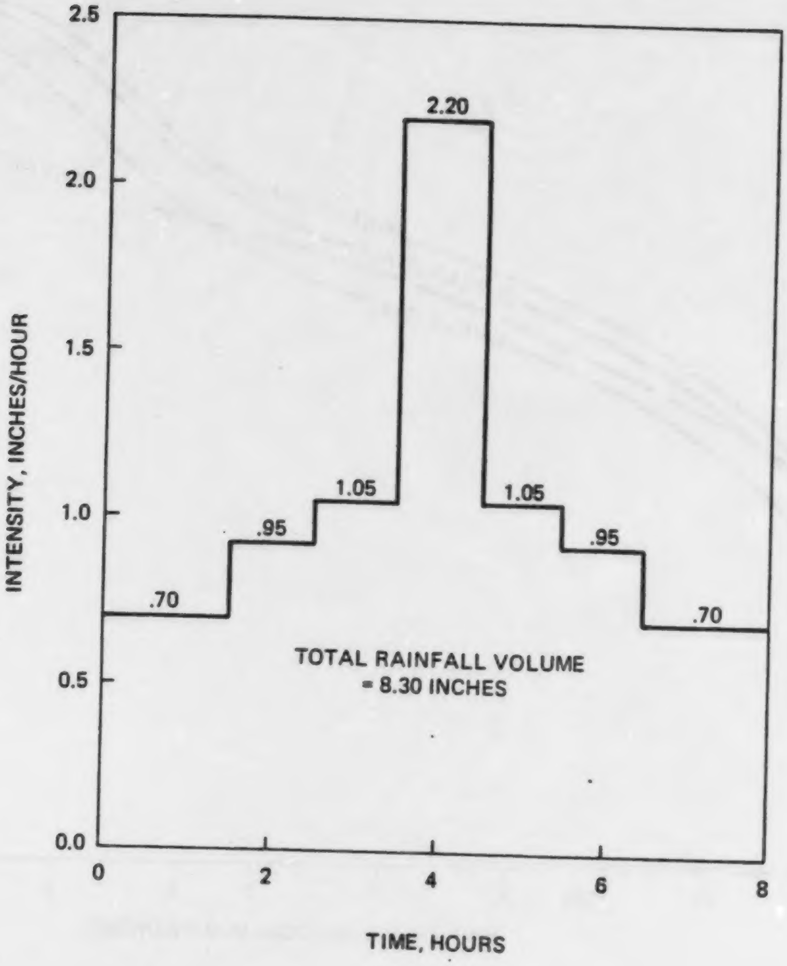

FIG. 7-2 RAINFALL HYETOGRAPH FOR THE FIVE YEAR, EIGHT HOUR DESIGN STORM

TABLE 7-1. FUTURE NO-ACTION HYDRAULIC RESPONSE OF OVERALL STUDY AREA TO DESIGN STORMS

Design Return Period (years)	5	25	50
Exceedance Probability (%)	20	4	2
Total Rainfall (inches)	8.6	12.2	12.3
Flood Volume (MG)	451	745	834
Runoff Volume (MG)	1,552	2,272	2,283
$\frac{\text{Flood Volume}}{\text{Runoff Volume}} \times 100$ (%)	29	33	37
Subareas that Flood	10	13	14
$\frac{\text{Flooding Subareas}}{\text{Total Subareas}} \times 100$ (%)	59	76	82

provided which would enable determination of design changes necessary to either prevent or reduce flooding.

Table 7-2 indicates that the largest amount of flooding, in decreasing order, occurred in the Forest Hills, Seaboard Coastline, North Tampa, Tampa Heights, Northwest Tampa and Hillsborough Avenue subareas. In Forest Hills, some of the overflow volume is fed to a Florida Department of Transportation retention facility for floodwater volume. As the Seaboard Coastline subarea is not highly developed, it is generally not a problem area for the City. The remaining major flooding areas are residential and would require remedial measures to alleviate flooding.

TABLE 7-2. SUMMARY OF FLOOD VOLUME FOR DESIGN STORMS

Subarea	Subarea Designation	Design Storm Frequency (years) 5	25	50
		Flood Volume (mgal)		
Forest Hills	FH	109	162	171
Seaboard Coastline	SC	75	109	113
North Tampa	NT	74	103	112
Northwest Tampa	NW	60	98	101
Tampa Heights	TH	60	93	100
Hillsborough Ave.	HA	49	78	87
West Tampa	WT	15	33	43
St. Joseph's	SJ	-	18	31
Seminole Heights	SH	6	15	21
Kirby Creek	KC	-	11	23
Sunshine Park	SP	2	9	13
Sulphur Springs	SS	1	5	7
Wellswood	WW	-	5	8
River Bend	RB	-	-	4

Based on the S/SWMM results, flood prevention values of storage volume, outflow capacity and runoff for the three storms in each subarea were derived. Table 7-3 presents these values for the five year event. Each flood prevention value was calculated as if the other two variables were maintained at existing or projected levels. Changes required in the system to achieve flood prevention are equal to the differences between the flood prevention values and the existing or projected values. Implementation of only one of these three options would control flooding in a given subarea.

The values presented in Table 7-3 can be used to evaluate only individual, single option solutions. For planning purposes, it is desirable to evaluate various combinations of these options to determine an optimal solution. To accomplish this, a flooding solution curve that contains all possible solutions was developed.

Derivation of a general flooding solution curve required definition of both a characteristic storage volume per unit area and a characteristic outflow capacity per unit area:

TABLE 7-3. FLOODING SOLUTION REQUIREMENTS FOR THE FIVE YEAR DESIGN STORM

Subarea abbrev.	Existing storage volume (mgal)	Flood prevnt. storage volume (mgal)	Existing outflow capacity(1) (mgal/hr)	Flood prevnt. outflow capacity(1) (mgal/hr)	Projected runoff (mgal)	Flood prevnt. runoff (mgal)
NT	13.7	87.3	1.0	12.1	96.3	22.6
SC	13.3	88.0	2.0	13.6	106.0	29.8
SS	2.7	3.4	10.7	11.3	53.2	50.9
SH	1.9	7.6	7.6	13.8	57.0	36.3
NW	32.9	93.1	1.0	8.6	102.0	41.8
HA	5.2	54.4	6.0	21.9	105.1	42.5
SP	1.8	4.0	18.0	19.8	83.0	75.3
TH	6.8	67.3	4.6	20.7	108.6	38.6
WT	3.5	18.8	9.2	21.2	87.0	42.5
Totals	79.5	424.0	60.1	143.0	798.2	380.3
FH(2)	17.2	125.8	5.9	28.0	178.6	61.8

1. Outflow capacity at downstream end of subarea, i.e. at river outfalls.
2. Assumes no overflow pumping capacity and is therefore not included in totals.

$$\text{Storage volume per unit area} = \frac{\text{Storage Volume}}{\text{Effective Runoff Area}}$$

$$\text{Outflow capacity per unit area} = \frac{\text{Outflow Capacity}}{\text{Effective Runoff Area}}$$

where, Effective Runoff Area is defined as (Runoff Area x Runoff Coefficient). Using these definitions and the flood prevention values of Table 7-3, a flood solution curve with the shape and features illustrated in Figure 7-3 can be determined for the design event. Each pair of storage and outflow on the flood solution curve represents one unique, flood prevention combination of the three factors which may be altered to reduce flooding (i.e. outflow capacity, storage volume and runoff). The advantage of using a flood solution curve is that any combination of flooding control can be evaluated.

Once the range of potential flood solutions has been defined, the costs of the practices required by the flood mitigation options can be estimated. Criteria used for determining these costs are shown in Table 7-4. Unless otherwise stated, the costs in this table were provided by the City. For planning purposes, the implementation needs and costs of stormwater controls can be estimated on a subarea or on an area-wide basis.

Management of Stormwater Quantity. Stormwater Quantity Management involves a preliminary planning phase and a detailed planning phase. The preliminary planning phase can be conducted by the City staff using the methodology presented here. It consists of incorporating information included in the SMP for a particular subarea and developing a preliminary range of

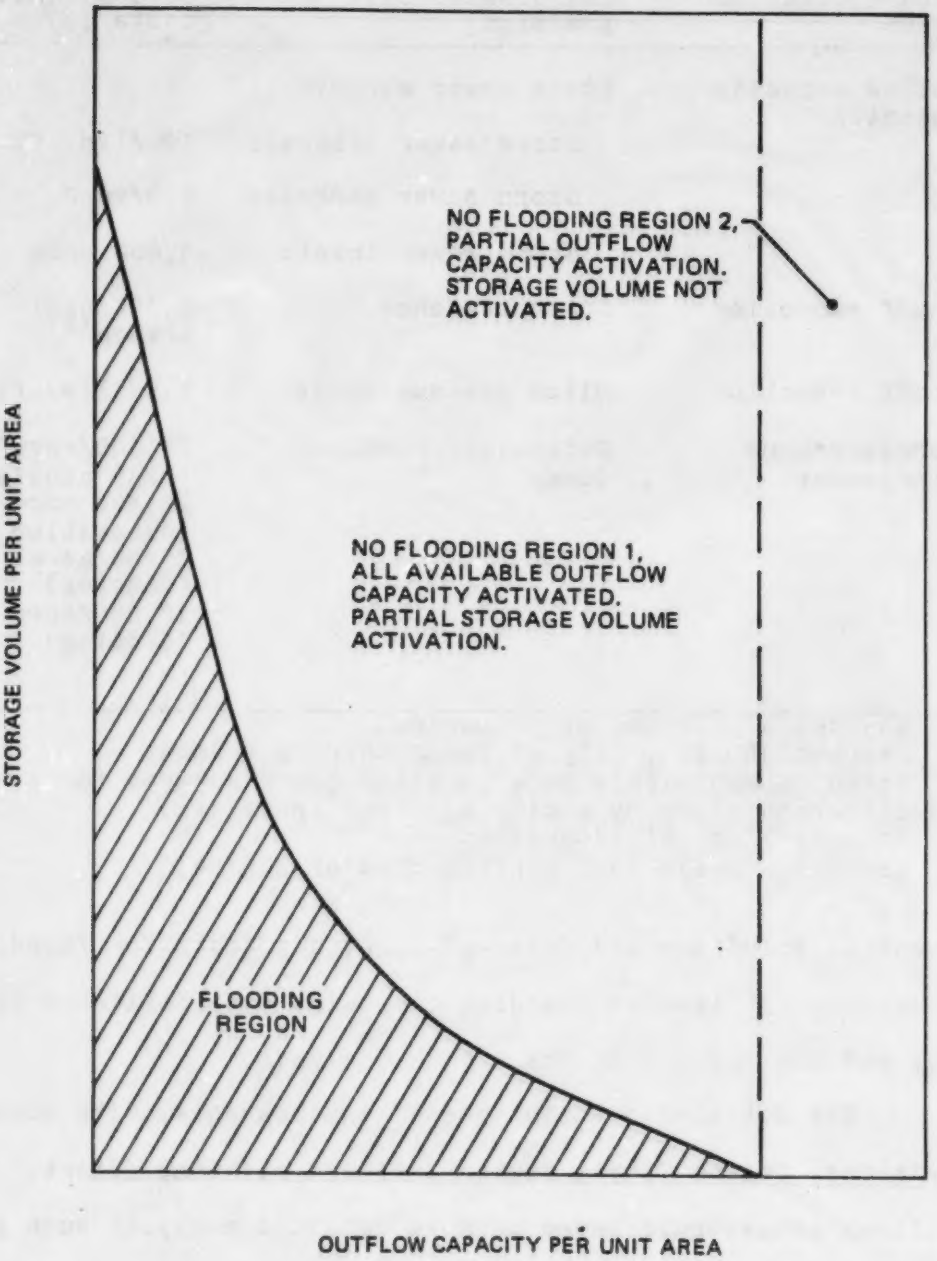

FIG. 7-3 CHARACTERISTICS OF THE GENERALIZED FLOOD SOLUTION

TABLE 7-4. SUMMARY OF IMPLEMENTATION PRACTICE COSTS

Flood mitigation option	Implementation practice	Capital unit costs ($)
Outflow capacity expansion	Storm sewer works:	
	Storm sewer laterals	100/lin. ft.(1)
	Storm sewer manholes	814/each
	Storm sewer inlets	1,800 each
Runoff reduction	Drain trenches	3,170/drain trench(2)
Runoff reduction	Ditch systems/swales	5.69/lin. ft.
Storage volume enlargement	Detention/retention ponds	25,000/acre(3) (land acquisition) 21,800 acre(4) (excavation) 2,900/acre(5) (fencing) 12,600/acre (sodding)

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1. Assumes a diameter of 54 inches.
2. Assumes 150 ft., City of Tampa uniform trench.
3. Based on appraisals made in River Grove subarea for a different study by a City approved appraiser.
4. Assumes 3 ft. of excavation.
5. Assumes average pond surface area of 3 acres.

potential solutions and order-of-magnitude costs for flooding solutions. It ties in flooding complaint data collected by the City and the results of the S/SWMM analysis.

The detailed planning phase is comparable, with some revisions, to the City's master drainage planning effort. It utilizes assessments based on more detailed analyses such as the SWMM.

Stormwater Quality Management

Effective stormwater management planning in the City of Tampa must take into account the quality of stormwater and its effect on the lower Hillsborough River. Stormwater quality management objectives are mainly a function of the needed reductions of stormwater pollutant load to the lower Hillsborough River from the drainage area. The following information is therefore required:

1. Identification of potential problem pollutants in stormwater with respect to water quality standards.
2. Selection of a design storm which will have the worst impact on the receiving water quality.
3. Estimation of design storm loads for potential problem pollutants.
4. Analysis of the impact of design storm loads for potential problem pollutants on the receiving water.
5. Determination of needed stormwater load reduction in the lower Hillsborough River.
6. Development of a stormwater quality management approach.

The information presented in this section allows the quality aspects of stormwater management to be taken into account in preliminary and detailed planning.

Potential Problem Pollutants. Table 7-5 compares the lower Hillsborough River water quality criteria with the quality of stormwater and of the River. BOD (through its effect on DO),

TABLE 7-5. COMPARISON OF LOWER HILLSBOROUGH RIVER WATER QUALITY CRITERIA WITH STORMWATER AND RIVER DATA

Parameter	Unit	Criteria	Mean Conc. of Pooled Runoff Data	River Data
BOD_5	mg/l	shall not cause DO depressions violating standards	14(1)	1-20
COD	mg/l	-	51	7-106
TSS	mg/l	settle to form putrescent deposits or a nuisance; float as debris or scum; produce a turbidity nuisance; <29 NTU's above background; depth of compensation <10% change.	40(1)	1-71
Nutrients:				
Ortho P	mg/l	man-induced nutrient load shall not cause imbalances in natural populations	0.14	0.08-0.93
Total P	mg/l		1.38(1)	0.15-0.95
Nitrate	mg/l		1.89	0.001-0.940
Nitrite	mg/l		0.02	0.000-0.022
TKN	mg/l		1.24	0.15-5.83
Ammonia-N	mg/l	0.02	0.38	0.044-0.77
Organic-N	mg/l		0.99	0.38-5.6
Total-N	mg/l		1.66	0.71-6.10
Fecal Coliform	MPN/100 ml	monthly avg = 200 10% >400 daily avg. = 800	24,800(1)	40-26,500
Total Coliform	MPN/100 ml	monthly avg. = 1,000 20% >1,000 never >2,400	156,200(1)	300-600,00

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(1) Potential Problem Pollutant

TABLE 7-5 (Continued). COMPARISON OF LOWER HILLSBOROUGH RIVER WATER QUALITY CRITERIA WITH STORMWATER AND RIVER DATA

Parameter	Unit	Criteria	Mean Conc. of Pooled Runoff Data	River Data
Aluminum	ug/l	1,500		50-530
Cadium	ug/l	5	2.49	-
Copper	ug/l	15	49(1)	3-57
Lead	ug/l	50	57(1)	4-26
Zinc	ug/l	1,000	59	4-9.0
Arsenic	ug/l	50	9	-
Barium	ug/l	-	35	-
Chromium	ug/l	50	8.4	-
Iron	ug/l	300	545(1)	-
Manganese	ug/l	-	14	-
Mercury	ug/l	0.1	6.9(1)	-
Nickel	ug/l	100	13	-

(1) Potential Problem Pollutant

suspended solids, total phosphorus, total and fecal coliform, copper, lead, iron and mercury exist in stormwater at concentrations which may violate the River criteria. Of these, copper, lead and iron are not expected to violate criteria because they require only minor dilution with River water. Other parameters such as mercury and coliform bacteria require much more dilution to satisfy the water quality criteria, and therefore are expected to be in violation at some locations in the River. For some stormwater pollutants, background River concentrations (due to other sources as well as stormwater) already violate the criteria. For these parameters, it was determined whether reduction of stormwater loads alone would achieve the criteria.

Selection of Design Storm. The adverse impact of a storm event on the receiving waters is associated with the washoff of surface solids caused by the storm. For a series of design storms, the shortest duration, lowest intensity storm that will still wash off most of the surface solids will have the most adverse impact on the receiving waters because it will have both maximum loads and concentrations. The design series from which the worst case event was selected was described in Chapter 4. The relative response of these storms was investigated by routing them with SWMM through the five monitoring basins, which had been calibrated against the discrete data as described in Chapter 4. SWMM simulation indicated that all of the design storms will wash off most of the surface solids.

Given that the pollutant load between storms changed little, the design storm of least total rainfall will produce the highest mean pollutant concentrations. Thus, it was concluded that the one-month one-hour design storm has the worst impact on the River. This storm has a total rainfall of 0.66 inches.

However, design storm consistency with State stormwater permitting requirements must also be considered in design event selection. The Florida Department of Environmental Regulation

(FDER) Stormwater Rule requires retaining the runoff from the first one inch of rainfall for areas larger than 100 acres. Several of the design storms had approximately one inch of total rainfall. Of these, the one-month two-hour storm impacted the River the worst. This storm is second to only the one-month one-hour storm in terms of River impact. For these reasons, this analysis was continued with the one-month two-hour event.

Design Storm Pollutant Loads. After establishing the one-month return period, two-hour duration storm as the water quality design event, pollutant concentrations that are the average of the composite, flow weighted concentrations from the five monitoring basins were determined. These concentrations were expected to be fairly representative of average concentrations for the overall study area. They are:

TSS = 77 mg/l

BOD_5 = 21 mg/l

Fecal Coliforms = 31,100 MPN/100 ml

The design event was then routed through the overall study area with these concentrations using the S/SWMM. Table 7-6 summarizes the results of this simulation. Runoff occurred for up to eleven hours after the storm, attenuating the impact on the River. None of the subareas flooded during this event.

Receiving Water Impacts. After quantifying the worst case of stormwater flows and loads to the lower Hillsborough River, their impact on the River was analyzed with the WQON model (2).

TABLE 7-6. DESIGN STORM POLLUTANT LOADS BY SUBAREA

Subarea Abbrev.	Runoff Volume, mgal	Hours of runoff	TSS, lbs.	BOD_5, lbs.	Fecal Coliforms MPNx10^{12}/100ml
FH	20.7	4	13,302	3,636	24
SJ	12.8	2	8,234	2,250	15
TH	12.6	3	8,090	2,211	15
SC	12.3	6	7,895	2,158	14
HA	12.2	2	7,826	2,139	14
KC	12.1	2	7,783	2,127	14
NW	11.8	11	7,594	2,075	14
NT	11.1	11	7,173	1,960	13
UT	10.1	2	6,506	1,778	12
WT	10.1	2	6,481	1,771	12
RB	9.8	2	6,301	1,722	12
SP	9.6	2	6,178	1,689	11
DT	8.6	2	5,552	1,517	10
WW	8.5	2	5,478	1,497	10
SH	6.6	2	4,247	1,161	8
SS	6.2	2	3,960	1,082	7
OG	4.7	2	2,991	818	5
Total	179.8		115,591	31,591	210

The parameters coliform, BOD/DO, solids and metals are primarily of interest. Modeling of coliform and BOD/DO was performed. The complex interactions of solids and metals between particulate and dissolved phases of the water column and sediments was not modeled.

A dry weather, low flow river condition with no freshwater release from the dam was modeled with stormwater inflow commencing at low water slack. The impact of the stormwater pollutant load is most critical for these River hydraulics.

A profile of fecal coliform concentrations one tide cycle after the end of the storm is compared with initial conditions in

Figure 7-4. The criteria for average daily fecal coliform count is also shown in this figure. Clearly, violations of fecal coliform concentrations occur throughout most of the River for at least one day after this storm event.

The DO profile one tidal cycle after the start of the storm is compared with initial conditions in Figure 7-5. This comparison leads to the conclusion that, in reaches of the River initially below about 4 mg/l, the stormwater has little effect on the DO profile. In reaches of the River initially above 4 mg/l, the stormwater causes a 1 to 3 mg/l depression in DO levels. For these reaches, the BOD of the stormwater serves to decrease River DO.

Analysis of available vertical DO profiles suggested that the DO violations during dry periods are due to the long-term exertion of benthic demand on an insufficiently flushed estuary. If this situation is corrected, the River should return to nonviolation DO levels during dry periods. Stormwater may cause violation conditions depending on what the dry weather River profile will look like once benthic demand is removed.

Management of Stormwater Quality. Based on the results presented in this section, the following issues regarding stormwater quality were incorporated into the City of Tampa's SMP:

1. Receiving water objectives for stormwater
2. Ranking of problem areas
3. Ranking of stormwater load reduction techniques

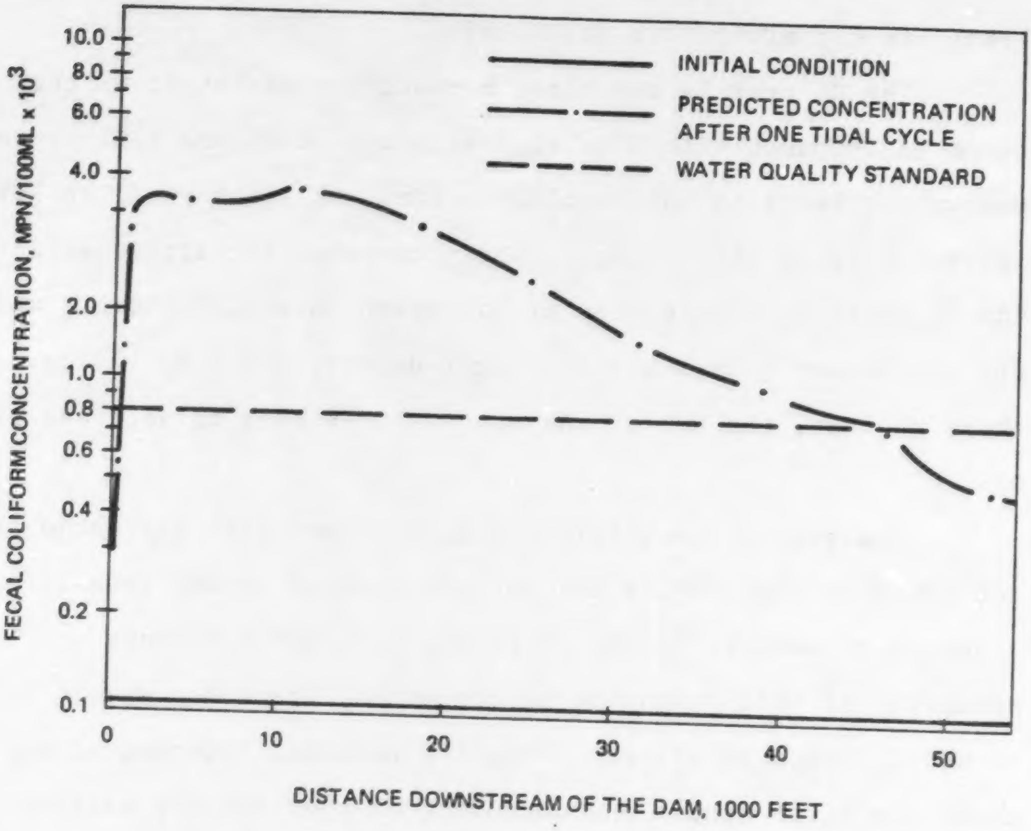


FIG. 7-4 FECAL COLIFORM PROFILES ALONG RIVER FOR WQON DESIGN STORM SIMULATION

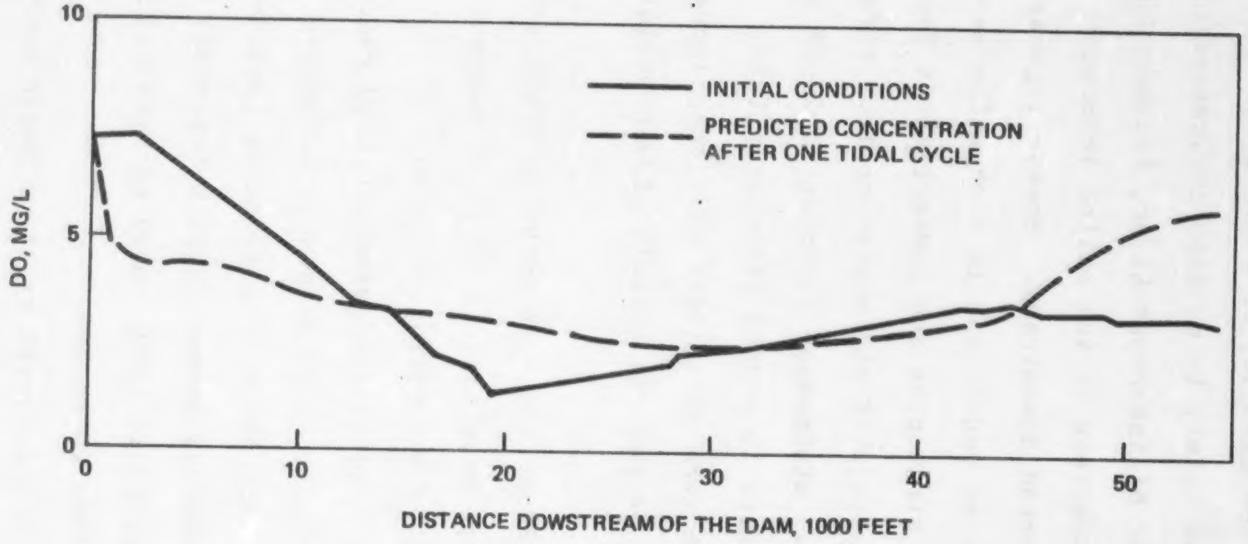


FIG. 7-5 DO PROFILES ALONG RIVER FOR WQON DESIGN STORM SIMULATION

If nothing else were changed, reasonable reductions in stormwater loads would not result in attainment of all water quality criteria. For example, DO would still occur at violation levels. This is due mainly to outside influences upon the quality of the lower Hillsborough River, including Bay water being transported upstream by the saline intrusion and Reservoir water being transported downstream. However, general strategies regarding desired load reductions in stormwater were considered. These strategies are summarized in Table 7-7.

Based on analysis of stormwater control effectiveness in Chapter 4, potential stormwater flooding controls were ranked in terms of their ability to reduce stormwater load. Earlier in this Chapter, these were classified as either increased capacity, increased storage, or reduced runoff. Their pollutant removal is summarized below:

1. Runoff Reduction - Achieved by drain trenches and ditches; results in a 30 to 70 percent removal rate (hydraulic and pollutant load).
2. Storage - Retention/detention pond removal rates can vary from 10 to 90 percent; effective in removal of particulate phase of pollutants (solids, metals) but not dissolved phases (BOD, nutrients); attenuates peak hydraulic load; does not generally result in reductions in volume.
3. Capacity - Achieved by storm sewer works; does not result in reduction in pollutant load; should only be used in instances where other methods not feasible.

TABLE 7-7. STORMWATER LOAD REDUCTION STRATEGIES

	Parameters		
Issue	Bacteria	BOD-DO	Solids[1] & Metals
Major Source	stormwater	Hillsborough Bay	stormwater, upstream drainage basin, and Bay
Correction Required to Meet Water Quality Standard	collection and disinfection of over 99 percent of flow	removal of saline intrusion and benthic demand	some removal of metals from all sources[2]
Possible City of Tampa Strategy	remove or detain stormwater when possible	build dam, dredge bottom, maintain effective freshwater flow	remove or detain stormwater when possible

1. Solids do not violate standards, but metals do.
2. Required removal of mercury would be far greater.

Both the identification of quality problems and objectives and the development of quality solutions should be incorporated into preliminary planning. In addition, during detailed planning, the recommended plan for the basin must conform to the receiving water objectives as summarized in Table 7-7.

Stormwater Permits

The State of Florida Administrative Code Chapter 17-25 sets forth a permitting system for stormwater discharges. The permit requires specific levels of stormwater control which result in certain removal or treatment of stormwater flows and loads. The permit requirements were evaluated and related to the strategies proposed to deal with stormwater flooding and pollution.

Florida Stormwater Permit Regulation. The primary reason for the regulation of stormwater discharge by the State is for the protection of receiving water quality. The regulation basically requires retention or detention (with filtration) of the runoff from one inch of rainfall, or one-half inch of runoff for areas less than 100 acres. The regulations establish which facilities must obtain stormwater discharge permits, and define the State concept of stormwater treatment. In addition, a list of exemptions to the permit regulations is put forth. These are given in Table 7-8.

If the Applicant can demonstrate conformance to one of the conditions in Table 7-8, the State will issue the permit. If not, the State will consider the following factors:

a) whether best management practices are used,
b) whether the public interest is served,
c) the efficiency and cost of alternative controls,
d) whether water quality benefits are reasonably related to costs, and
e) whether reasonable operations and maintenance provisions have been made.

Impact of Permit on Stormwater Quantity. The major permitting issues which relate to stormwater quantity are:

1. The effect of retention or detention of runoff from 1 inch of rainfall.
2. The effect of 72 hours of detention or retention time as a basis for design.

TABLE 7-8. EXEMPTIONS TO STORMWATER PERMIT REGULATION

1. One single family dwelling, duplex, triplex or quadriplex which is not part of a larger development.
2. Single family residential projects of less than 10 acres total area with less than 2 acres impervious surface, provided that it is part of an approved master drainage plan.
3. Facilities consisting entirely of swales
4. Facilities for agricultural land.
5. Facilities for silvicultural land.
6. Facilities which provide retention, or detention with filtration, of the runoff from the first one inch of rainfall, or from the first one-half inch of runoff for areas less than 100 acres.
7. Facilities consisting of grassed waterways designed to percolate runoff from a 3-year 1-hour design storm within 72 hours.
8. Modification or reconstruction of an existing system which will not serve new development and which will not increase pollutant load.

---

3. The impact of a 3-year 1-hour design storm for design of grassed waterways.

Current City design practices were evaluated with respect to the above. For retention/detention ponds, City design requirements are more stringent than DER requirements. For grassed waterways, DER design requirements are more conservative than existing design practices.

The overall step-by-step approach to stormwater management planning in Tampa was expanded to incorporate stormwater permitting considerations. The complete stormwater management planning approach, as shown in Figure 7-6, introduces the evaluation of permit impacts into preliminary planning, the incorporation of permit requirements into detailed planning and obtaining the stormwater permit into the design and construction phase.

Case Study - Sunshine Park

The three phases of the SMP shown in Figure 7-6 were demonstrated for the Sunshine Park test case area. This was done to serve as a model for future City efforts.

As indicated in Figure 7-7, the test case area is located to the east of the lower Hillsborough River within the Sunshine Park subarea. It is approximately 436 acres in size. The test case area is primarily residential, has three outfalls to the River and no storage ponds.

Preliminary Planning Phase. During preliminary planning, the test case area was identified as a frequent source of flood complaints. Use of the general flooding solution curve indicated that any of the following single solutions would alleviate flooding:

1. Increase storage volume by 1.5 mgal (88 percent of the existing value).
2. Increase outflow capacity by 1.7 mgal/hr (18 percent of the existing value).
3. Reduce runoff by 6.8 mgal (14 percent of the projected value).

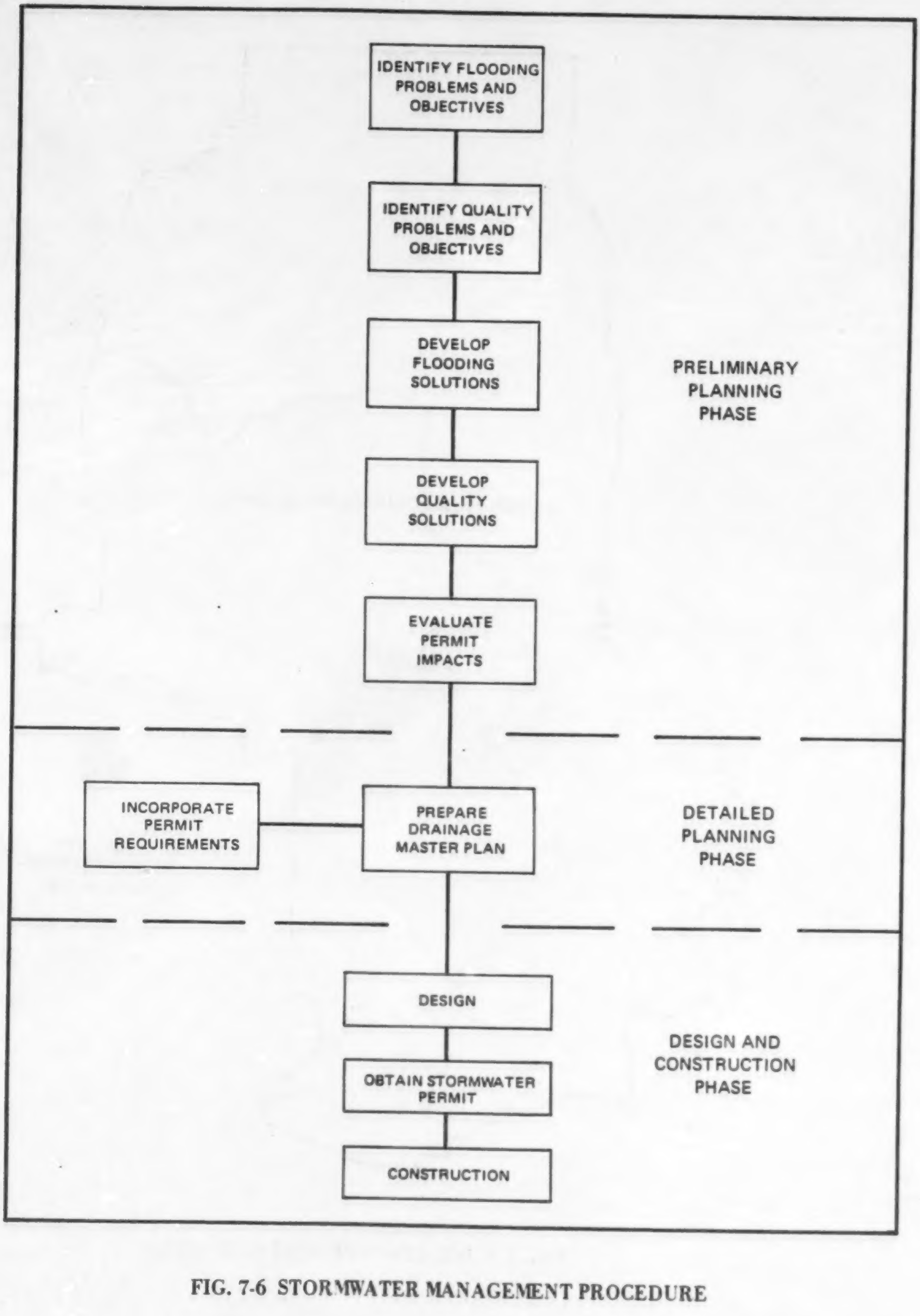

IDENTIFY FLOODING PROBLEMS AND OBJECTIVES
IDENTIFY QUALITY PROBLEMS AND OBJECTIVES
DEVELOP FLOODING SOLUTIONS
DEVELOP QUALITY SOLUTIONS
EVALUATE PERMIT IMPACTS
PRELIMINARY PLANNING PHASE
INCORPORATE PERMIT REQUIREMENTS
PREPARE DRAINAGE MASTER PLAN
DETAILED PLANNING PHASE
DESIGN
OBTAIN STORMWATER PERMIT
CONSTRUCTION
DESIGN AND CONSTRUCTION PHASE

FIG. 7-6 STORMWATER MANAGEMENT PROCEDURE

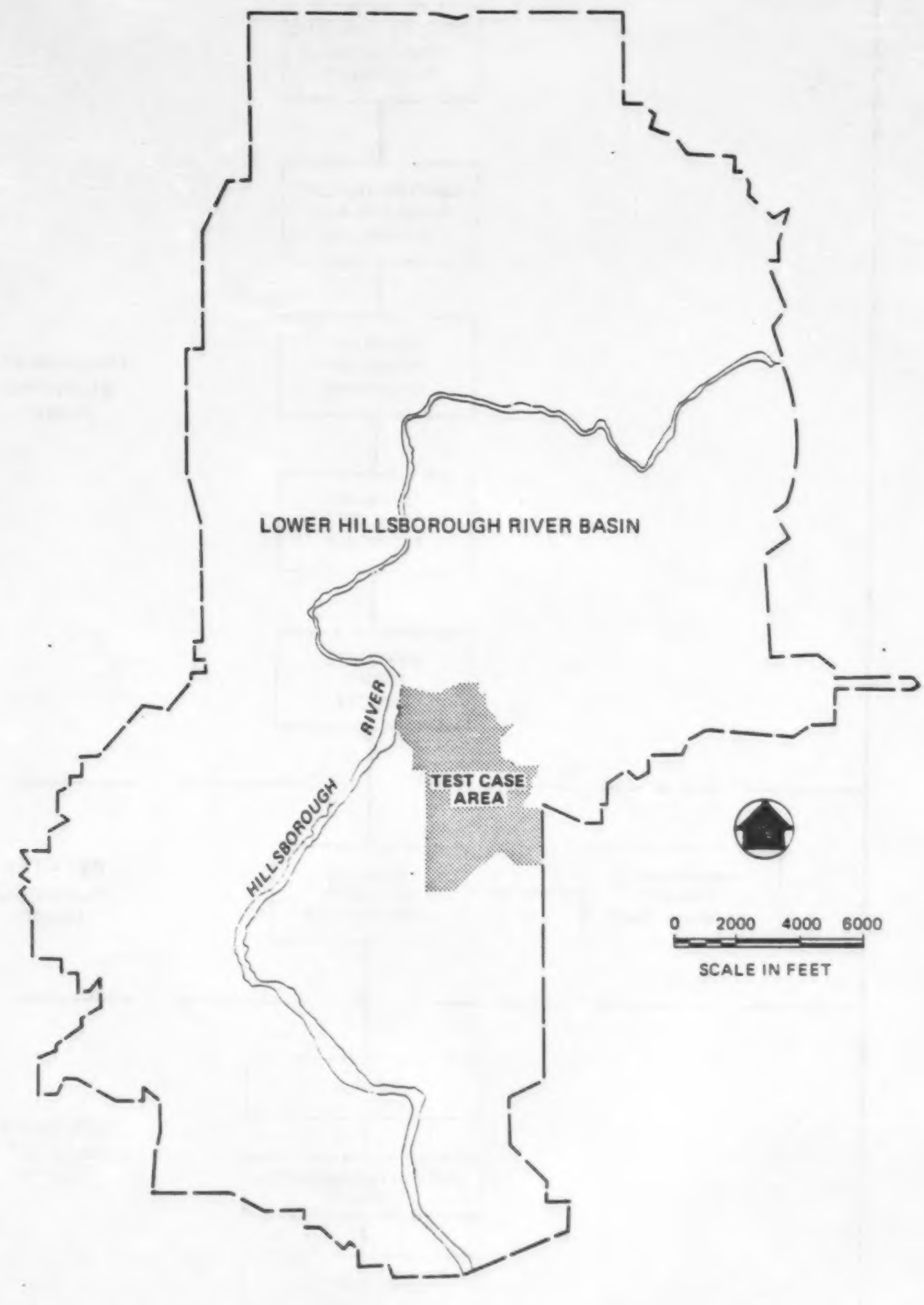

LOWER HILLSBOROUGH RIVER BASIN
HILLSBOROUGH RIVER
TEST CASE AREA
0 2000 4000 6000
SCALE IN FEET

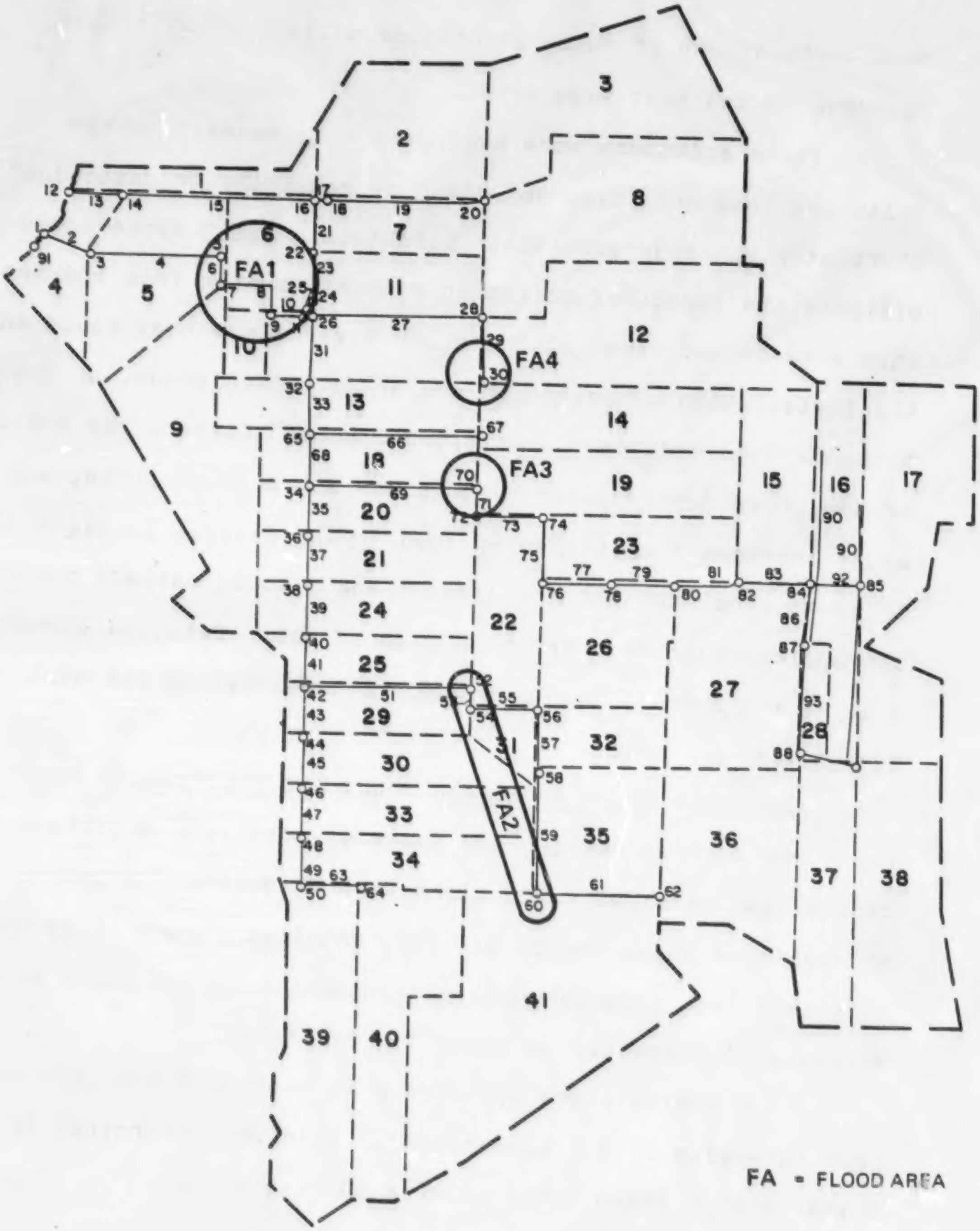
FIG. 7-7 LOCATION OF TEST CASE AREA

Many combinations of these strategies would also alleviate flooding in the test case area.

These solutions were evaluated with respect to the pollutant load reduction objective of retaining or detaining stormwater whenever possible. Solutions 1 and 3 above would mitigate the impact of pollution from stormwater from the test case area on the lower Hillsborough River since they would entail the installation of detention and/or retention ponds, or drain trenches and/or ditches. Solution 2 would increase the capacity of the storm sewer system through new storm sewer works, and would increase the stormwater load to the River. In light of this, it is clear that Solutions 1 and 3 would satisfy the State stormwater rule while Solution 2 would not. Detailed planning will be required to evaluate which alternative is the most reasonable.

Detailed Planning Phase. Once the Preliminary Planning Phase was carried out and the range of available solutions determined, detailed evaluations were conducted. This was accomplished using the SWMM. This entailed discretizing the test case area into subcatchments and schematizing the storm sewer system into elements, as shown in Figure 7-8.

The design storm hyetograph in test case simulations had a peak intensity of 4.5 inches/hour. This was determined from the 5 year design storm curve using a 20 minute interval. This time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FA = FLOOD AREA

FIG. 7-8 TEST CASE AREA DISCRETIZATION AND STORM SEWER ELEMENTS WITH EXISTING FLOOD AREAS PREDICTED BY SWMM

interval was selected so that peak rainfall intensities would accurately represent local conditions.

SWMM simulation of the future no-action, test case condition during the design storm event resulted in 5.1 mgal (about 8 percent of total runoff) of floodwater. Almost 50 percent of this flood volume occurred in Flood Area 1 as indicated in Figure 7-8, which coincides with the major flood complaint district, demonstrating the predictive capability of SWMM.

Flood control practices simulated in the test case area included detention ponds, drain trenches and storm sewer laterals. Ditches were not simulated as their installation seemed undesirable in a highly residential area. It was found that over 7 acres of detention ponds were required to prevent flooding. Acquiring this amount of land for this use would be costly in the thickly settled test case area. The drain trench solution showed that about 200 drain trenches would eliminate flooding. The 200 manholes required to retrofit these trenches to may not be readily available in the test case area. This left storm sewer laterals as the most feasible single solution. The requirements of this solution are given in Table 7-9.

The outflow capacity solution will not reduce the stormwater pollutant load to the River and will not meet the DER Permit Requirements. Therefore, to ameliorate receiving water impacts and comply with the Permit as much as possible, the City should install some drain trenches, which will remove some of the stormwater pollutant load. Such a mixed solution could be optimized with the general flood solution curve.

TABLE 7-9. CHARACTERISTICS OF ADDITIONAL STORM SEWER LATERALS REQUIRED IN TEST CASE AREA TO PREVENT FLOODING

Flood Area	Capacity (mgal/hr)	Length (ft)	Location
1	3.0	1780	Lousiana, from intersection with Highland, to Ola to Violet to River
2	1.6	2330	Suwanee, from intersection with Buffalo, to Genesee to Florida to Genesse, at intersection with Highland
3	0.6	1780	Hilda from Highland to Florida
4	0.3	380	Florida from Osborne to Louisiana

REFERENCES

1. City of Tampa, City of Tampa Preliminary Subsector Plans, Tampa 2000 Land Use Element, 1980.

2. Dailey, J.E. and D.R.F. Harleman, Numerical Model for the Prediction of Transient Water Quality in Estuary Networks, Ralph M. Parsons Laboratory for Water Resources and Hydrodynamics, Report No. 158, Department of Civil Engineering, Massachusetts Institute of Technology, October, 1972.

CHAPTER 8

SUPPLEMENTS TO THE STORMWATER MANAGEMENT PLAN

General

As part of the City's Stormwater Management Plan, several investigations and evaluations were conducted to supplement the major aspects of the plan which were described in the previous Chapter. The purpose of these supplements is to provide input to the City on various ongoing activities and procedures which affect the management of stormwater in the City of Tampa. These include review of City:

1. Drainage ordinances
2. Design criteria
3. Drainage system design practices
4. Operations and maintenance activities
5. Stormwater and receiving water monitoring

The findings of these specialized investigations are described in this Chapter.

Drainage Ordinances

In this supplement, existing City ordinances that affect drainage are reviewed and suggested revisions to the ordinances are outlined. Historically, the City ordinances dealing with stormwater have only been modified on a piecemeal basis. Therefore, it is recommended that the City adopt a unified drainage ordinance.

Several sections of the City Codes address various aspects of stormwater drainage. These sections are summarized in Table 8-1.

TABLE 8-1. CITY OF TAMPA CODES WHICH ADDRESS STORMWATER DISCHARGE

Code	Chapter	Area of Influence
Subdivision	35	Drainage utilities for subdivisions
Zoning	43	Physical factors which impact drainage (i.e. lot size, setback, density)
Buildings and Building Regulation	45 - Article XV	Earth moving and its impact on drainage
	Article XVI	Flood damage control
Boats, Docks and Waterways	10	Material which is allowed to be discharged to storm sewers
Plumbing	50	Connections of stormwater piping

A single, unified drainage ordinance which comprehensively covers the City's policy on drainage would assist City Staff, developers and design professionals involved with drainage projects in Tampa. Such an ordinance should establish public policy through clear policy statements. Specific procedures and design criteria should be collected and contained in a Drainage Engineering Manual. Sections of existing ordinances should be incorporated into the unified ordinance and the Drainage Engineering Manual. These sections are discussed below. A preliminary outline for the ordinance is shown in Table 8-2, and for the Manual in Table 8-3.

TABLE 8-2. OUTLINE OF UNIFIED DRAINAGE ORDINANCE

ARTICLE	TITLE
I.	SCOPE
II.	DEFINITIONS
III.	STATEMENT OF OBJECTIVES
IV.	ESTABLISHMENT OF FLOOD PRONE LANDS CLASSIFICATIONS
V.	FLOOD PLAIN DISTRICT USES
VI.	SPECIAL USE PERMITS
VII.	NONCONFORMING USES
VIII.	HYDROLOGIC AND HYDRAULIC STUDIES
IX.	IMPROVEMENTS REQUIRED
X.	COST OF DRAINAGE IMPROVEMENTS
XI.	EROSION AND SEDIMENTATION CONTROL
XII.	GRADING AND DRAINAGE PLANS REQUIRED
XIII.	MAINTENANCE
XIV.	SUBDIVISION PLATS
XV.	BONDING
XVI.	REPEAL OF CONFLICTING ORDINANCES
XVII.	VIOLATION AND PENALTY
XVIII.	APPEALS
XIX.	EFFECTIVE DATE

TABLE 8-3. OUTLINE OF DRAINAGE ENGINEERING MANUAL

I. AUTHORITY
  1.1 Drainage Ordinance(s)
  1.2 State Law(s)
  1.3 Inter-Agency Cooperation

II. REVIEW PROCESS
  2.1 Review Process Description with Schematics
  2.2 Review Punch List(s)
  2.3 Construction Completion Certification

III. DRAINAGE POLICY
  3.1 Minimum Drainage Requirements
  3.2 Comprehensive Design Requirements
  3.3 Cut-off Criteria for Stormwater Management Plans
  3.4 Design Philosophy
  3.5 Water Quality Standards
  3.6 Minimum Treatment Standards
  3.7 Overdrainage/Water Construction
  3.8 Loss of Historic Basin Storage Capacity
  3.9 Flood Plain Encroachment
  3.10 Upstream/Downstream Effects
  3.11 Maintenance of Drainage Capacity During Construction
  3.12 Building Flood Elevations
  3.13 Resolution of Utility Conflicts
  3.14 Curbs and Gutters
  3.15 Pavement Slope

IV. PLANS, REPORTS AND STUDIES STANDARDS
  4.1 Drainage Calculations Reports
  4.2 Hydrologic and Hydraulic Investigations
  4.3 Plans and Design Drawings
  4.4 Standard Work Sheets

V. DESIGN POLICY
  5.1 Rainfall Data
  5.2 Evapotranspiration
  5.3 Antecedent Conditions
  5.4 Design Storm(s)
  5.5 Time of Concentration
  5.6 Runoff
  5.7 Off-site Discharge Regulation
  5.8 Pass-Through Drainage Regulation
  5.9 Receiving Water Stage
  5.10 Percolation
  5.11 Infiltration
  5.12 Soil Borings
  5.13 Storage
  5.14 Routing
  5.15 Minimum Elevations
  5.16 Erosion Control
  5.17 Grassing and Mulching

TABLE 8-3 (cont.). OUTLINE OF DRAINAGE ENGINEERING MANUAL

VI. DESIGN CRITERIA
- 6.1 Drainage Easements
- 6.2 Access/Maintenance Easements
- 6.3 Detention/Retention
- 6.4 Underground Exfiltration
- 6.5 Swales
- 6.6 Open Channels
- 6.7 Pipes
- 6.8 Discharge Structures
- 6.9 Control Structures
- 6.10 Weirs
- 6.11 Orifices
- 6.12 Flumes
- 6.13 Inlets
- 6.14 Junction Boxes
- 6.15 Manholes
- 6.16 Underdrains

VII. DESIGN STANDARDS INDEX

The Subdivision Ordinance contains a number of requirements related to stormwater drainage. These requirements should be reflected in the unified drainage ordinance.

The Zoning Ordinance establishes the City's policy on setback requirements for construction within flood prone areas. The general policy statement is appropriate for an ordinance but specific identification of lots and parcels would be more appropriate in the Drainage Engineering Manual. The current list of flood prone lots should also be included in the manual.

The Buildings and Building Regulations Ordinance contains two sections which address excavations affecting stormwater and flood damage control. One allows the excavation, grading, or filling of lands within the City provided that the individual undertaking these activities secures a permit from the City and that the proposed excavation will not adversely affect surface drainage. This policy statement is typical of many ordinances in Florida but is difficult to enforce as the public is generally not aware that fill cannot be placed on private property. This section also contains requirements for side slopes and fencing of drainage retention/detention ponds that is presented in a somewhat confusing format. These criteria should be clarified and placed in the Drainage Engineering Manual.

Section 45-400 of the Buildings and Buildings Regulations Ordinance contains the standard provisions for flood damage control mandated by the National Flooding Insurance Program. This section contains standardized criteria for communities desiring Federal Flood Insurance. Such criteria should be incorporated in the Drainage Engineering Manual.

The Plumbing Ordinance provides for the regulation of storm drainage plumbing within a building and of swimming pool discharges. While the direct effect of this regulation on the overall drainage program is minor, these requirements should be incorporated in the Drainage Engineering Manual.

The drainage sections of existing ordinances have established both policy and design criteria. As urban drainage problems have become better understood in the last decade, City administrators have developed policies for the design and construction of urban stormwater systems that are consistent with the rising complexity of the quantity-based issues and problems.

Conversely, many of the general policies and broadly stated objectives contained in the drainage sections of the existing ordinances tend to be specific to a number of classic drainage problems. A unified drainage ordinance would serve the City well through reviewing and revising the existing public policy statement and including new policy statements which address the complex drainage problems that have risen in the last decade and are compatible with the regulatory environment of the 1980s.

The drainage sections of the existing ordinances virtually exclude reference to water quality. With the evolving environmental concerns of the 1970s and the expanded understanding of urban runoff, more attention has been directed toward stormwater runoff quality. Notable within the regulatory environment is the development of the State of Florida's Stormwater Rule which requires the treatment of stormwater discharges.

The majority of the City of Tampa drains to the Hillsborough River both above and below the City water works dam. The Florida Department of Environmental Regulation has become increasingly concerned with the quality of this water body and the City is presently facing an involved permitting process for new stormwater outfalls to the river. In line with these developments, the City should consider adopting certain water quality goals and public policies in the unified drainage ordinance.

Design Criteria

This supplement to the City's SMP includes design criteria for the City to incorporate into the Stormwater Management Plan. The development of design criteria is in large part based upon the Tampa NURP Phase II Study (1). This consisted of an extensive field data collection program and subsequent analysis of the data collected. Specific criteria were developed for the following areas:

1. Rainfall Analysis,
2. Runoff Volume Calculations,
3. Water Quality Considerations,
4. Control Measures.

Rainfall Analysis. Until recently, a storm event with a 3 or 5-year recurrence interval has been primarily utilized by the City of Tampa as the basis of storm sewer design. However, with the increase in urbanization and development within the City of Tampa, the Department of Public Works has adopted a policy wherein a storm with a 25-year recurrence interval has been utilized as the minimum basis for calculating permissible offsite discharge.

Historically the rainfall intensity-duration curves prepared by the State of Florida and contained in the Florida Department of Transportation Drainage Manual have been used by the City to define design storms. During the Tampa NURP Study, design storms were developed from 22 years of record at the Tampa International Airport. These curves were developed from local data collected at the Tampa International Airport and include the intensity-duration relationships for storms with 1, 3, 6 and 12 month and 5, 25 and 50 year return frequencies. It was found that for durations greater than about 3 hours (this would include all storms used for flood protection), the Tampa NURP design storms had larger rainfall volumes.

In some cases it may be necessary to continue use of the Florida DOT rainfall curves because of regulatory requirements, the need to update historical calculations using comparable methods, and local familiarity. However, since the Tampa NURP design storms are based on local data and provide additional protection, the City should consider use of these design storms.

Runoff Volume Calculation. The recommended runoff volume calculation procedure is the Rational method for projects less than 200 acres and for all individual parcels. For larger systems and drainage basin master planning, a numerical modeling approach using SWMM (or a comparable single event model) is proposed. From extensive single event modeling using SWMM, a list of calibrated hydraulic coefficients was developed which can be used in future analyses by the City. For calculation of runoff volume on a long-term basis or for major drainage basins, a simplified stormwater modeling calculation using a model such as S/SWMM can be performed. Runoff coefficients and other model inputs were developed during the NURP study which can be used for such modeling.

Water Quality Considerations. Another consideration in storm sewer system design is to mitigate and reduce water quality impacts on the receiving waters. Tampa has sponsored the Receiving Water Study in the lower Hillsborough River to investigate these considerations and augment the Tampa NURP Study. The goal of the Receiving Water Study was to develop a model which accurately simulates observed hydraulic and water quality characteristics in the lower Hillsborough River and then use this model in the analysis of impacts upon the receiving water due to urban runoff. A series of storms derived from the intensity-frequency-duration curves during the NURP study was analyzed in order to determine the relative impacts on the receiving waters. A storm which washed off the greatest pollutant load while providing the least dilution was sought. Based on this analysis, a water quality design storm corresponding to a 1-month 2-hour event with 1 inch of rainfall was selected. The duration corresponds to the most frequent storm duration in Tampa and the volume is consistent with the DER requirement of retention of runoff from 1 inch of rainfall on-site.

As mentioned, part of the analyses conducted during Phase II (1) was extensive single-event and long-term modeling of stormwater quantity and quality. To predict water quality in small systems and individual parcels, concentrations representative of quality for the entire lower Hillsborough River Basin developed in Phase II should be used. For larger studies (i.e. master drainage planning), the SWMM can be used with default values modified pursuant to Phase II findings to better simulate Tampa runoff quality.

Control Measures. Proposed retention/detention pond criteria incorporate current trends in pond design such as consideration of seasonal high water table and inclusion of a littoral shelf and allow for municipal design limitations such as fencing, maintenance and access easements and minimum/maximum grades. A flow chart was prepared which allows the designer to make certain basic design decisions and arrive at the appropriate set of design criteria.

Essentially, little change has been recommended from the existing design standards for drain trenches.

Open bottom inlets have a current City design standard that has not been adhered to during installation. Specific design criteria were prepared based upon the test results described in Chapter 3 and City standards for design.

Ditches were tested for hydraulic extraction as described in Chapter 3 and design criteria were prepared. Where water table conditions permit, it is recommended that ditches be undercut 6 inches in order to enhance both hydraulic reduction and pollutant removal characteristics.

Drainage System Design

In this supplement, a methodology for design of major and minor drainage systems is described. Existing City drainage design practices have been revised and updated to incorporate findings of the Tampa NURP study and other current design standards. The methodologies incorporate both stormwater quantity and quality considerations, coordinate design procedures with desired stormwater load reductions, and aim towards achieving consistency with current DER stormwater permitting regulations.

Current City Drainage Design Practices. Existing City design practice is based upon an internal memorandum (2). The peak runoff rate for storm sewer design is calculated using the Rational method. Storm intensity is determined using the Florida Department of Transportation (FDOT), intensity-frequency-duration (IFD) curves for Zone 6, Tampa. Storm duration is determined from the Velocity of Runoff Chart in the FDOT Drainage Manual.

Detention ponds are designed based on the 25 year IFD, FDOT curve for Zone 6 Tampa using a 24 hour duration unless maximum capacity requirements are obtained under less duration. Retention ponds are designed based on the 50 year IFD, FDOT curve using a 24 hour duration.

Distinction Between Major and Minor Systems. For purposes of this supplement, major system design refers to applications which use SWMM, and are often greater than 100-200 acres. Minor system design uses the Rational method, and applies mainly to areas less than 100-200 acres. The demarcation between major and minor systems is dependent upon two concerns. One is the inherent limitation in the Rational method approach. It is invalid to use this method in drainage systems greater than about 200 acres (3) because non-peak flows, which this method does not account for, become more important as basin size increases. Thus a lower bound is established for the size of a major system where more detailed analyses such as SWMM are required. For areas less than 100 acres, the Rational method is clearly valid and can be used. For areas between 100 and 200 acres, both methods can be considered. The major system analytic approach involves the SWMM, which is relatively expensive in that it requires access to a main frame computer and specialized engineering expertise. The following factors must be evaluated to determine the need for SWMM modeling:

1. The value of the system area and properties adjacent to it that may be affected by flooding in the area.
2. The agency, whether city or private, being held responsible for the design.

For low value district drainage systems of less than 200 acres, the City may deem it appropriate to use the Rational method. However, in high value districts, SWMM could be used for City designed systems. SWMM can always be used, even for very small areas, for which it will still provide accurate results. However, City staff designing small systems and private developers may not require this level of design detail.

Major System Drainage Design. The major system drainage design has preliminary planning, detailed planning and design and construction phases. Each phase is discussed below.

Preliminary Planning Phase - The preliminary planning phase facilitates stormwater management on a large scale. During this phase, the flood prevention requirements of the major system are estimated. If the major system area corresponds to a subarea of the lower Hillsborough River basin as designated in Chapter 2 of this Study, then these requirements have already been calculated. Evaluation of such requirements allows prioritization of the major system with respect to the other major systems within the City that may require drainage system work. The following steps are involved in preliminary planning:

1. Identify flooding problems,
2. Identify quality problems,
3. Develop flooding solutions,
4. Develop quality solutions,
5. Analyze stormwater permit effects.

Detailed Planning Phase - The detailed planning phase involves development of the detailed Master Drainage Plan for the basin. Major design considerations, assuming all detailed drainage system data are available for the basin, are development of a design storm and the stormwater modeling analysis.

The Tampa NURP intensity-frequency-duration curves developed in Chapter 2 can be used to develop design storms. If required, the FDOT curves can also be used, although they are less conservative. A time interval of approximately 20 minutes should be used in order to capture realistic peak intensities. This interval could be adjusted based on site specific data.

Designing new systems or improvements to existing systems in accordance with the five year curve will provide an adequate level of protection, consistent with many other urban areas. However, if the budget can accommodate providing high value districts with more protection, this could be considered.

Using the SWMM for stormwater modeling enables storm sewer systems to be "fine tuned" and allows comparison of alternatives so that State stormwater permitting requirements are met. Ideally, improvements to the existing system should be incorporated into the SWMM until it predicts no surcharging. SWMM ensures compatibility of existing and proposed systems as it models their integration in detail.

The quality impacts of the implementation practices can be evaluated based on the removal rates established for various flood control practices during Phase II (1).

Design and Construction - After all planning studies are completed and a recommended drainage plan has been developed, final design and specifications can be prepared for the drainage system. Most of the information required to perform the final design has been developed through SWMM modeling. This includes pipe, pond and ditch sizes, slopes and locations. The necessary information to apply for a stormwater permit will also have been generated. Other information needed for final design can be developed through standard City practices.

Minor System Drainage Design. Minor system drainage design follows current City drainage design using the Rational method. Only minor changes in use of this method are suggested. Changes include incorporation of Tampa NURP Phase II findings(1) concerning runoff coefficients, intensity-frequency-duration relationships and storm protection. Other minor design details are based on practices in use at Metcalf & Eddy, Inc.(4).

Quality considerations in minor system design are the same as those for major systems.

Operations and Maintenance

This supplement provides input to the City's Stormwater Management Plan on a methodology to review existing and proposed systems and establish an operations and maintenance program which considers the required repairs and replacement activities for optimal quantity and quality control. The overall O&M problem and the projected activity and service levels were examined and the latter two were evaluated as to their consistency with the stated goals and objectives of the program. Recommendations are presented for maintenance procedures for the stormwater management practices tested during Phase II(1).

The O&M Section is the largest of the groups in the Stormwater Management Division with approximately 98 positions and a 1983 budget of about $2.5 million. The O&M Section is responsible for three major services which operate and maintain the drainage infrastructure for the City: ditch and pond maintenance, storm sewer repair and construction, and storm sewer cleaning and street sweeping.

The size and scope of the O&M problem faced by the Stormwater Management Division is partially quantifiable and can be divided into known and unknown problems. The known problems have already been identified and can be quantified and evaluated. Generally, the known aspects of the O&M problem deal with the physical inventory which the Department has maintained such as ditches, pipe systems, ponds, and the drainage aspects of the road network. Table 8-4 breaks down the Tampa O&M Division's jurisdictional drainage network.

TABLE 8-4. COMPOSITION OF EXISTING TAMPA DRAINAGE NETWORK

Ditches - Major	26 miles
- Minor	200 miles
Retention/Detention Ponds	90 ponds
Stormwater Pump Stations	30 stations
Pipes - Greater than 24 inches	176 miles
- Up to 24 inches	365 miles
Inlets	21,200 units
Manholes	10,600 units
Culverts	7,700 units
Curb & Gutter	1,226 miles
Streets - Residential	978 miles
- Commercial	330 miles

The unknown problems include a number of factors which cannot be described and/or quantified easily. These include meteorological variations, groundwater fluctuations, budget and funding availability, resolving joint City and County drainage problems and assessment of system-wide structural conditions.

Furthermore, routine maintenance is currently funded inadequately and it is not clear whether the Stormwater Management Division can adequately compete with other City-wide priorities in the future. These issues, along with an essentially non-existent replacement maintenance program and a minimal capital improvement program, are leading the City to a point of resorting to a separate funding source, based on service charges.

Section Goals and Objectives. This section discusses the goals and objective/condition standards which have been developed for each of the three services of the O&M Section.

The first of the three services focuses on the maintenance of the existing ditch system and retention/detention ponds. The goal of this service is to maintain proper hydraulic flow line elevations and to eliminate all health and safety hazards in open drainage systems. Eleven objective/condition standards have been developed to support the generalized statement of the goal. Eight of the standards are specific for ditches and three of the standards were developed for ponds.

The standards which were developed specifically for ditch systems address the routine maintenance aspects such as mowing, silt accumulation, washout repair, and trash accumulation as well as the safety related issues of locations of ditches in the safe recovery zone of major road segments and adequate access for maintenance.

The second service deals with the maintenance of the existing storm sewer system and includes the stormwater pump stations. The goal is to maintain proper hydraulic flow line and eliminate health and safety hazards related to inadequate drainage systems. Thirteen objective/condition standards have been developed to assist in the work activities related to achieving this goal. Seven of the standards are related to repair and reconstruction of the existing system, three standards apply to new construction required to bring the existing system into compliance, and the remaining three standards are related to stormwater pumping systems.

The last service, storm sewer cleaning and street sweeping, is targeted at maintaining hydraulic capacity of the existing system. The goal is to maintain proper flow line elevations, satisfy water quality objectives, and provide aesthetically acceptable street gutters. Eleven objective/condition standards have been developed to support this broad goal.

Storm sewer cleaning is the more labor intensive consideration and is addressed by six specific standards. The focus of four of these standards is the accumulation of materials in the pipe which reduces hydraulic capacity, one standard addresses field response to flooding complaints, and the sixth standard addresses the development of a standard for two new pollution abatement devices.

Activity/Service Level Evaluation. The evaluation of the activities undertaken in 1983 was divided into four categories: services planned, projected budget expenditures, goal attainment, and long-term projects.

The 1983 Work Program and Budget contains an inventory of existing facilities which need to be operated and maintained and projects the annual work quantity within each activity which should be accomplished. While many of the annual work quantity units are not readily comparable with the inventory measure, seventeen of the forty-three activities do allow a direct comparison. Comparison of objectives and planned work activities indicates a trend in the work program which will have long-term, adverse ramifications: the piping components of the storm sewer system are being neglected. While both major and minor ditches are being routinely maintained and while street sweeping is efficiently proceeding, the major maintenance aspects of the pipe system such as necessary excavation and construction, reconstruction, slip lining, and large pipe cleaning are not being accomplished.

In order to reverse this trend and prevent the collapse of the drainage infrastructure, substantially larger sums of money must be allocated for the reconstruction, repair, and cleaning of the storm sewer pipe network.

Of the $2.5 million 1983 O&M budget, $1.7 million is for labor while $0.8 million is for equipment and material costs. The ditch and pond maintenance service utilized 33.0 percent of the total budget and 37.1 percent of the total labor.

The following Table presents the allocation of the resources budgeted for ditch and pond maintenance in terms of manpower, total dollars, percent of the labor budget, and percent of the total Division O&M budget.

TABLE 8-5. O&M BUDGET ALLOCATED TO DITCH AND POND MAINTENANCE

	Total Labor (man days)	Total Dollars Required	% of Labor Budget	% of Section Budget
Major Ditches	275	47,317	5.7	1.9
Minor Ditches	7,834	718,959	86.2	28.4
Detention Ponds	726	46,774	5.6	1.9
Other	244	21,045	2.5	0.8
Total	9,079	834,095	100.0	33.0

The budget allocated for the construction/reconstruction of storm sewers and stormwater pumping activities made up nearly half of the total budget. The resources allocated for this service were $1,130,977 which represents 44.7 percent of the total budget and 39.6 percent of the total labor. Approximately half of these resources were utilized in the repair and reconstruction of the existing storm sewer system.

Another technique available to analyze the proposed work program and budget is to evaluate the frequency for which goals have been attained. Data from 1982 have been reviewed for this purpose. For any objective/condition standard, there are four possibilities which exist for assessment which include: over-achieved, achieved, missed, and unknown. The goal achievement results for each of the three services are presented in the following Table:

TABLE 8-6. GOAL ATTAINMENT IN O&M (1982)

	Ditch and Pond Maintenance	Storm Sewer Repair and Construction	Storm Sewer Cleaning and Street Sweeping
Over-Achieved	2	1	0
Achieved	2	1	4
Missed	7	6	5
Unknown	1	5	2
TOTAL	12	13	11

These findings for achievement of goal levels suggest that the following alternatives should be considered: either the overall output of the work program must be increased (through increases in productivity or increases in funding) in order to achieve the target levels, or the target levels are unrealistic in light of available funding and administrative constraints and should be revised.

Evaluation and Control Mechanisms. From an operational standpoint, services can be delivered based on a fixed service level that is tied to a fiscal year budget target. It would be beneficial to the Department and to the City in general if the time lag between the assessment of productivity and activity levels and the establishment of budgetary levels was reduced.

The establishment of drainage as an enterprise fund with its own revenue sources will provide for increased opportunities to accomplish necessary O&M functions but will also necessitate the expansion of staff and the motor pool. This expansion will place additional demands on the evaluation and control mechanisms which are operative. However, the City's service level analysis program is functionally sound and should persevere.

A mechanism should be developed for coordination within the Division between the O&M Section and the Planning and Design Sections. This coordination would allow for the exchange of information and would benefit all Sections. Information obtained in the field through adverse experiences can be passed on to the Design and Planning Sections which would benefit their procedures and provide a more "hands-on" basis for their activities. Similarly, activities which are to be undertaken by the Planning and Design Sections can be discussed with the O&M Section for the mutual benefit of all groups.

Long-Term Monitoring

Current City monitoring activities were reviewed and recommendations were made to improve or revise these activities based on the findings of the Tampa NURP Study. This is explained in the following sections, for stormwater related monitoring activities (rainfall, stormwater, pond levels, and groundwater levels), and receiving water monitoring activities.

Stormwater Related Monitoring. An optimal network of nine rain gages was developed. Six of the gages would be owned and operated by the City of Tampa and the remaining three gages would be operated by other entities (NOAA, WTVT, and USF). This network would leave the City with two reserve rain gages which can be utilized for basin specific studies or for substitution purposes when any of the six existing City gages are being repaired. Four relocations of existing City gages would be required to implement the optimized rain gage network.

A precipitation quality monitoring program was also developed for implementation in the City of Tampa. The objective of this program is to document areal and seasonal variability and precipitation quality characteristics. It is recommended that three gages in the rain gage network be utilized for this program.

For stormwater runoff characterization, it is recommended that the available flow monitoring equipment from the Tampa NURP Study be deployed in the field in those basins which will be studied in the next fiscal year in order to collect real-time data for use in the basin modeling process. No more than four units should be employed at any given time. In addition to documenting flow in existing systems for purposes of basin studies, the equipment can be utilized to evaluate the hydraulic function and effectiveness of major detention/retention facilities owned and operated by the City.

It is also recommended that the City of Tampa institute a stormwater runoff collection and analysis program which would serve two beneficial purposes. First, the additional water quality data can be combined with the original sixty data points in an effort to improve the definition of the probable concentrations for the selected parameters. Second, the samples can be selected in conjunction with basin studies for different types of land uses which will document specific water quality characteristics for the basin(s) under study.

It is recommended that major facilities which act as retention ponds for larger drainage systems be monitored throughout the wet season. These larger ponds, which have no natural drainage outlet and consequently must be pumped, represent a significant flooding hazard. Monitoring of these lakes during the wet season, and during the ensuing dry season when the water level falls and the pond recovers to its normal levels, will provide an advance warning as to when emergency pumping may be required to prevent flooding and supplement available data regarding pond hydraulic behavior.

In order to accumulate knowledge on groundwater levels and soil characteristics in these identified areas, it is recommended that the City expand its existing network of 76 monitoring wells to cover additional areas of the City which are not well covered.

Receiving Water Monitoring. It was determined that the quarterly water quality data collection as well as sediment sampling programs are adequate at the present time for continuation of monitoring in the River. Other more detailed synoptic data collection programs are not considered necessary at the present time. Their recent undertaking has provided information on estuarine behavior that is not likely to change significantly unless specific River management alternatives are implemented.

The following suggestions, which aim at providing better information on the background conditions in the River and therefore lead to a better understanding of the data collected, were outlined for these programs.

The following suggestions are made for revision of the quarterly data collection program to capture the effect of freshwater flow releases from the Reservoir on water quality, and improve data completeness, presentation, and ease of analysis:

1. Conduct each quarterly survey during a specific River condition. Perform one after several weeks of very low release and another during a period of high release. The other two could be performed during an average freshwater release and after a storm event preceded by dry weather.
2. Include the freshwater discharge from the Reservoir and Sulphur Springs and the rainfall antecedent conditions in the quarterly summary sheets.
3. Increase the number of in-situ measurements of salinity and dissolved oxygen to at least 5 vertical points.
4. Add in-situ profiling of River velocity, also at least five vertical points.
5. Include chlorophyll a and turbidity measurements in the quarterly data collection.
6. Take a sample of the Reservoir water release to identify River boundary quality input.

7. Modify current measurement station locations to include the four NURP stations.

Evaluation of sediment quality also provides information for long-term analyses. Sediment quality data should be obtained twice yearly, during both low and high freshwater flows since bottom conditions are influenced by freshwater flow. In addition to the parameters currently analyzed, volatile content, organic carbon content, copper, lead and zinc are useful indicators of bottom conditions and should be routinely collected and analyzed.

If additional funds are available for continuous monitoring of the lower River, it is suggested that several meters be placed at one station to identify vertical variation of salinity and DO.

Finally, data from any other ongoing data collection programs by other Agencies should be obtained for comparison with City data.

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Respectfully submitted,

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